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EXPEDIENT UPGRADING OF EXISTING STRUCTURES FOR FALLOUT PROTECTION

by

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This study was conducted in support of the Defense Civil Preparedness Agency's (DCPA) Crisis Relocation Planning (CRP) program in which existing structures will be upgraded to provide fallout shelters for a relocated population. A demonstration test was conducted in which a residential dwelling was upgraded by placing soil against the walls and on the roof of the structure. The shelter was large enough to house 80 people. Upgrading was accomplished partially by hand labor and machinery. The test (Continued)

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20. ABSTRACT (Continued).

showed that a conventional structure could be upgraded and that the shelter occupants using tools and materials found in most homes could if necessary upgrade their shelter during the expected 2- or 3-day period of crisis relocation preceding a nuclear attack.

Several roof systems were tested and others were analyzed based on previous test results for overloads that occur from upgrading. A system of added supports was developed that would allow steel open-web joist roofs to be upgraded. Flat wood roof systems were found to have sufficient overload capacity to be upgraded without added support. Upgrading in all cases consisted of adding 100 to 120 psf of mass to the roof. Some concrete roof systems such as the flat plate were found unacceptable unless the quantity of upgrading material was reduced. The two-way slab roof could be safely upgraded without modifications.

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PREFACE

This study, sponsored by the Defense Civil Preparedness Agency under Project Order No. DCPA01-74-C-0233 (March 1974) and Project Order No. DCPA01-75-C-0286 (April 1975), was conducted by the U. S. Army Engineer Waterways Experiment Station (WES) during the period March 1974 through June 1976.

The study was conducted under the general supervision of Mr. W. J. Flathau, Chief of the Weapons Effects Laboratory (WEL), and Mr. J. T. Ballard, Chief of the Structures Division, WEL. The report was prepared by Mr. W. L. Huff, Structures Division.

Commanders and Directors of WES during conduct of the study and preparation of the report were COL G. H. Hilt, CE, and COL J. L. Cannon, CE. Technical Director was Mr. F. R. Brown.

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FIGURES

CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI)
UNITS OF MEASUREMENT

U. S. customary units of measurement used in this study can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
inches	25.4	millimetres
feet	0.3048	metres
miles (U. S. statute)	1.609344	kilometres
square feet	0.09290304	square metres
cubic yards	0.764555	cubic metres
pounds (mass)	0.4535924	kilograms
pounds (mass) per inch	178.5797	grams per centimetre
pounds (mass) per foot	1.488164	kilograms per metre
pounds (mass) per square foot	4.882428	kilograms per square metre
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
pounds (force) per square inch	6.894757	kilopascals
degree (angle)	0.01745329	radians

EXPEDIENT UPGRADING OF EXISTING STRUCTURES FOR FALLOUT PROTECTION

CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

Military strategists presently believe that there is little possibility of a surprise nuclear attack on the U. S. and that, if a nuclear war were to occur, it would be preceded by a period of intense crisis. Recent Soviet civil defense manuals stress the importance of protecting the civilian population by dispersal and excavation in conjunction with the use of protective shelters and individual means of protection. Soviet planners base evacuation plans and preparation on the belief that there will be at least 3 days to complete evacuation and for the evacuees to protect themselves. With the present U. S. transportation system the evacuation of the population from high-risk to low-risk target areas should be much simpler.

This study is part of the present effort of the Defense Civil Preparedness Agency (DCPA) to guide planners in developing an American capability for an additional civil defense option called "Crisis Relocation Planning" (CRP). CRP would complement - not replace - plans to protect the population in place, in cities, and elsewhere. Under CRP, people are moved from target areas to outlying or host areas over a period of 2 or 3 days. The target areas would not be abandoned but manned by critical occupation workers who would keep the vital services functioning. Relocation would be to public and private buildings in outlying areas and, if necessary, to shelter. The movement would be in response to a developing international crisis and not to the launching of an attack.

There is insufficient shelter space in many host areas for even the area residents; therefore, additional shelter spaces should be provided.

These shelters could be made available through construction of expedient shelters such as the designs developed at the Oak Ridge National Laboratory (Reference 1) or by expedient upgrading of space in existing structures. The critical occupation workers would have to be provided blast as well as radiation shelters. This study addresses the problem of upgrading existing structures to provide fallout shelters for the relocated population.

1.2 OBJECTIVE

The objective of this study was to develop upgrading procedures, demonstrate their feasibility, and investigate the strengthening requirements for upgrading existing structures in host areas to provide shelters for the relocated and host residents. The upgraded shelters in this study were required to have a minimum protection factor (PF) of 40, as this is required for present in-place fallout shelters.

1.3 SCOPE

A single-story residential dwelling was upgraded to provide shelter space for approximately 80 people. The upgrading required placing soil to a height of 6 feet[1] against the exterior walls of the house and to a depth of 12 inches over the entire roof. Soil placement was accomplished by a variety of methods that included hand labor, machinery, and a combination of both. Data obtained from these various methods were then extrapolated to provide time and manpower requirements for upgrading the shelter by any of the methods used or to predict the time and effort required for upgrading other types of structures. Information obtained on the strength of the walls and roof was used to determine the modifications necessary for structures of similar construction to support the added mass necessary for radiation shielding.

An expedient single-family shelter was constructed by tunneling under the slab-on-grade foundation of the house used in the upgrading

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[1] A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 6.

demonstration test previously discussed. The shelter was large enough for a family of four and was constructed using tools and materials found in most homes.

Eight roof support systems were analyzed to determine adequacy to support a superimposed load of 100 psf (12 inches of soil cover). Three of the systems were statically tested in the laboratory to verify load-carrying capacity and to evaluate use of additional supports to increase load-carrying capacity. These were 10- and 28-foot-span open-web steel joist roof support systems and a 12-foot-span wooden roof support system. Data on the static load capacity of several other roof support systems were obtained from previous test results.

CHAPTER 2

[redacted]

UPGRADING A RESIDENTIAL DWELLING

A single-story brick veneer house with a concrete slab foundation was obtained from the Department of Housing and Urban Development (HUD) to be upgraded to a multiple-family fallout shelter. The house represented a lightweight structure that when upgraded would provide an insight into the problems associated with moving and placing the additional mass required for upgrading, methods of covering openings, restoration when the crisis has past, and structural difficulties that could apply to other type structures.

The house located in a hillside subdivision approximately 4 miles north of Vicksburg, Mississippi, was among a group of houses that suffered severe foundation problems during heavy rains in the latter part of 1973. It was in good condition structurally, but was vacated about 6 months prior to the test. During that time all of the windows were broken and considerable damage was done to the inside by vandals.

Since the foundation was a concrete slab-on-grade, DCPA suggested trying to construct a single-family shelter under the concrete-slab foundation. This type shelter would provide the option of a private shelter similar to the basement shelter. The shelter was constructed by tunneling under the slab foundation so as to determine structural adequacy of the foundation to span over the shelter and difficulties encountered in digging.

The multiple-family shelter was constructed by upgrading the entire house to a category 2 (PF of 40 to 69) fallout shelter, which was accomplished by adding mass to the walls and roof of the house. The added mass consisted of soil piled against the exterior walls and on the roof of the house.

2.1 DESCRIPTION OF TEST STRUCTURES

Figure 2.1 is an aerial view of the subdivision in which the test house was located. The house had a 1042-square-foot heated area and a

345-square-foot single-car garage. The floor plan is shown in Figure 2.2; front and rear views of the house are shown in Figure 2.3a and b, respectively.

The foundation consisted of a 4-inch-thick concrete slab-on-grade reinforced with 6 by 6-6/6 welded wire fabric. A beam 16 inches deep by 12 inches wide reinforced with four 1/2-inch-diameter reinforcing bars was located along the perimeter and down the centerline of the slab.

The exterior walls were of conventional construction consisting of 2 by 4 studs at 16 inches on center with the inside and outside faces of the wall covered with 1/2-inch-thick dry wall and builders board, respectively. The brick veneer facing of the outside wall was placed with a 1-inch air gap between the builders board and the brick. The brick veneer wall was tied to the 2 by 4 studs with metal ties. A cross section of the wall construction is shown in Figure 2.4.

The roof of the house was supported by prefabricated roof trusses spaced at 24 inches on center. Over the main portion of the house, the trusses were fabricated from 2 by 4's with the top chord being a 2 by 6. The roof trusses were fabricated entirely of 2 by 4's over the garage area. The two truss types are shown in Figure 2.5. The roof sheathing was 3/8-inch-thick plywood covered with roofing felt and asphalt shingles.

2.2 SINGLE-FAMILY SHELTER

2.2.1 Shelter Construction. The single-family shelter was constructed partially under and alongside the slab foundation. Workmen were instructed to build a shelter under the slab foundation approximately 4 feet wide by 8 feet long with a ceiling height of 4 feet. The access trench was to be only as large as necessary alongside the foundation to provide working space to dig under the foundation. Final dimensions of the shelter and access trench were 4 feet wide by 7 feet 1 inch long by 4 feet 9 inches high, and 4 feet wide by 5 feet 8 inches long, respectively. A cross section of the completed shelter is shown in Figure 2.6.

Figure 2.7 shows the completed access trench and shelter under the

slab. The cover over the access trench was fabricated by placing interior doors from the house over the trench and covering them with soil for radiation protection. Most of the interior doors in the house were of the hollow-core type, constructed from two pieces of thin veneer held together by strips of cardboard. Two layers of doors were used for the trench cover to insure support of the earth cover. Two damaged louvered doors were used for the bottom layer and two hollow-core doors were used for the top layer (Figure 2.8).

A soil roll on which the doors were supported was constructed around the sides of the trench to keep surface water from rains from running inside the shelter. The soil roll was constructed by placing a 4-foot-wide strip of polyethylene alongside the access trench with approximately 2 feet of the polyethylene draped into the trench. Soil was placed on the polyethylene and the free edge was folded over the soil. Two by four's spanned the trench crosswise to help support the door roof.

Before placing the 16 inches of soil on the roof of the trench, the doors were covered with polyethylene to keep them dry. A soil roll, as described above, was used at the entrance end of the shelter to hold the soil cover in place (Figure 2.9). A rain shield attached to the edge of the roof and a layer of polyethylene over the soil covering the doors aided in keeping the shelter dry.

The entranceway was closed with a single interior door covered with soil-filled pillowcases. The door was supported on short pieces of 2 by 4's that also provided a skid (Figure 2.10) allowing opening and closing of the entranceway. With the entranceway closed, the door was raised the thickness of the 2 by 4's providing some ventilation for the shelter. Additional ventilation could be provided by propping up the edge of the entranceway and by digging a small ventilation shaft at the other end of the shelter. Figure 2.11 shows the interior of the shelter.

2.2.2 Construction Effort and Protection. A total of 8-1/2 yd^3 of soil was moved during the construction of the shelter. The actual time required to construct the shelter was 12-1/2 man-hours, plus

1 man-hour to construct the access trench cover, giving a total time of 13-1/2 man-hours. It is estimated that a man and his wife could construct the shelter in less than 16 hours.

Digging of the shelter revealed several unexpected problems. The location of the water and sewer lines under the slab foundation coincided with the location chosen for the shelter. The shelter location therefore was moved approximately 18 inches to avoid the lines. Initially, the access trench for digging the shelter was to serve only as the entrance. The trench was enlarged, however, in order to have adequate working space for one person to tunnel under the house and consequently was included in the shelter area.

The completed shelter contained a floor area of 50 square feet and provided a PF greater than 50. The PF of the shelter could be doubled by placing 8 inches of soil on the concrete floor slab over the shelter.

2.2.3 Unreinforced Footing Test. The perimeter footing and concrete floor slab were cast integrally with both containing reinforcing. In cold climates the perimeter footing of the house is separated from the concrete floor slab by insulation. The perimeter footing is placed deeper in the ground to get below the frost line and, in many cases, does not contain any reinforcing. The safety of digging under a foundation of this type to construct a single-family shelter as described above was questionable. Therefore, an analysis and load test of an unreinforced footing was conducted.

Figure 2.12 shows a typical cross section of a house with a concrete slab-on-grade and an unreinforced perimeter footing. In most cases, a short concrete block or concrete foundation wall extending above the soil line will be placed on the footing and support the aboveground outside walls of the house. This short foundation wall should aid the unreinforced footing considerably in spanning over a tunnel under the footing. This effect was omitted in the analysis and test in order to examine the worst loading case.

For the load test, an unreinforced footing 12 feet long by 16 inches wide by 8 inches thick was poured on top of a level section of ground at the U. S. Army Engineer Waterways Experiment Station (WES)

Weapons Effects Laboratory (WEL) Big Black Test Site. Concrete for the footing was 2500-psi compressive-strength design mix from a local ready-mix concrete company. Standard 6- by 12-inch test cylinders made at the time the concrete footing was cast were broken at 28 days with an average breaking strength of 2830 psi. Concrete blocks were stacked on the foundation to simulate the weight of an 8-foot-high exterior house wall. The blocks were placed in two layers 4 feet high for safety since mortar was not used. Figure 2.13 shows the footing with the 4-foot-high concrete block wall completed. A trench 4 feet wide and about 12 inches deep was dug under the center of the footing to simulate the entrance to an expedient fallout shelter. Figure 2.14 shows the footing and wall with the trench completed. The footing did not show any signs of distress while supporting the concrete block wall with a 4-foot-wide unsupported span in the center. Additional weight was added to the top of the wall as shown in Figure 2.15a and b. The weights were 1000-pound blocks of steel. With the 3000 pounds of additional weight, as shown in Figure 2.15b, the footing still did not show any signs of distress.

Assuming the footing to be acting as a beam 4 feet long and fixed at both ends, loading of the concrete blocks and 3000 pounds of steel weights would produce a maximum shear stress of 18.6 psi and maximum tensile stress due to bending of 111.6 psi. With this loading, the maximum shear and tensile stresses are approximately 10 and 140 percent, respectively, of the allowable values from the 1963 American Concrete Institute (ACI) Building Code. The maximum load carried by the footing during the test was approximately twice the loading that would be expected in actual use. Therefore, it was concluded that the expedient shelter constructed under the HUD house could be safely constructed under a house having an unreinforced footing and floor slab as shown in Figure 2.12.

2.2.4 Test Results and Conclusions. A total of 13-1/2 man-hours was required to remove 8-1/2 yd^3 of soil and cover the entranceway of the single-family shelter constructed under the slab-on-grade foundation of the house. A man and his wife not accustomed to manual labor should be able to construct the shelter during the expected 2 or 3 days

of crisis escalation using tools and materials found in most homes.

After the crisis has past, the house can be restored by filling the shelter with soil and hand tamping. The shelter could be made into a permanent fallout shelter by lining the walls and floor and adding a more secure entranceway.

The walls of under-the-slab shelters constructed in sandy soils may need to be shored to prevent sloughing. This could be a serious problem for shelters constructed under houses having an unreinforced perimeter footing where soil sloughing would increase the unsupported span of the footing. Tensile stresses in the unreinforced footing are directly proportional to the square of the span length. As an example, a 10 percent increase in span would produce a 20 percent increase in the tensile stress in the concrete. The addition of shoring at midspan of the unsupported section of the footing would be required. It is recommended that, for any unsupported span longer than 4 feet, a center support be provided whether the footing is reinforced or unreinforced.

The under-the-slab single-family shelter provides residents of host areas an alternate to the community shelter. By placing soil on the floor overhead for radiation protection, the single-family shelter could be constructed under a house having a conventional pier-beam-joist foundation. An example of this is shown in Figure 2.16.

2.3 MULTIPLE-FAMILY SHELTER

For this test, soil was placed against the exterior walls of the house 6 feet higher than the interior floor level and over the entire roof to a thickness of 4 inches. On one end of the house, the soil cover on the roof over a 14-foot-wide strip was increased in increments of 6 inches to a thickness of 24 inches on the front side and 18 inches on the back side. This was done to determine the load-carrying capacity of the roof system.

For the safety of the workers, all glass was removed from the house. The interior ceiling and fiberglass insulation in the attic were removed to allow better access to and visibility of the roof trusses during loading. Shrubbery around the house was also removed in order to place soil against the exterior walls.

Ideally, soil used to cover the shelter should come from a borrow area close enough to be hauled by wheelbarrows. Because of the small size of the lot on which the house was built and the erosion in the backyard, it was necessary to haul soil obtained from a site approximately 1/4 mile away in dump trucks. The soil was then placed by hand labor and machinery against the walls and on the roof of the house.

To upgrade many public or commercial buildings, it will be necessary to haul soil to the site due to the location of parking areas around the building, streets, and adjacent buildings.

2.3.1 Wall Upgrading. Soil was placed against four exterior walls altogether containing 10 windows and 2 doors. These openings were to be covered with undamaged interior doors laid horizontally across the openings; however, insufficient undamaged doors remained in the house. Two of the front windows were selected as typical and were covered with interior doors; the remaining openings were covered with 3/4-inch-thick plywood.

Figure 2.17 shows the plywood being placed over the floor-to-ceiling living room window. Small nails were used to hold the plywood in place, which could also have been accomplished by placing some soil against the plywood. Figure 2.18 shows the procedure used to cover the remaining windows. Soil was placed against the wall until it was just below the windows. An interior door that had been wrapped in plastic to keep it dry was laid horizontally across the window. Soil was then placed against the door holding it in place. When the level of soil was at the top edge of the door, another door was placed like the first door. The remaining openings were covered in a similar manner using plywood.

Entrance to the shelter was provided through the kitchen. Plywood was placed across the bottom 4 feet of the door (Figure 2.19a) and soil was placed against the wall to the top of the plywood. Wing walls along the sides at the top half of the opening were made by using an interior door that had been sawed in half (Figure 2.19b). Placing of the soil was then continued up the wall to the required height. The shelter entrance was 30 inches wide by 42 inches high.

Using a front-end loader, soil was placed against the front wall of

the house to a height of 6 feet above the interior floor level (Figure 2.20). The top of the soil berm was 1 foot thick. Allowing the soil to take on its natural slope, the base of the soil berm turned out to be 7 feet thick. The time required for placing soil with the front-end loader against the front wall of the house and estimations for covering all four walls of the house are contained in Table 2.1.

The soil berm against the inside garage wall was placed entirely by hand. It was dumped from trucks on the driveway and hauled with wheelbarrows to the inside of the garage. Labor used consisted of two men pushing wheelbarrows, two men filling the wheelbarrows, and two men shoveling the soil inside the garage. The time required to reach various levels of radiation protection was obtained by placing the soil against the wall in different stages. During the first stage, soil was placed 3 feet high for the entire length of the wall. The top of the berm had a thickness of 10 to 12 inches. The height of the berm was then increased in 1-foot increments to a final height of 6 feet. Figure 2.21 shows the completed wall. The amount of time and quantities of soil moved are shown in Table 2.1. Note particularly the decrease in soil placement rate as the height of the soil berm increased.

The interior garage wall consisted of builders board covered with sheetrock and consequently was not as strong as the other walls against which soil was placed. No provisions were made to prevent the absorption of moisture from the soil.

Soil was placed against the end and back of the house using a crane and clamshell bucket (Figure 2.22). This type of equipment was used because of the inaccessibility to the rear of the house and limited working room on the end of the house. Both areas could easily have been done by hand labor; however, sufficient information on hand labor had already been obtained from the work in the garage. After dumping the soil beside the walls with the clamshell, the soil was shoveled into place by hand.

<u>2.3.2 Roof Upgrading</u>. Prior to covering the roof with soil, the prefabricated trusses supporting the roof were inspected for defects. The joints of the truss were held together with sheet metal brackets

that had been punched to form 3/4-inch-long spikes on one side. In fabricating the trusses these brackets were placed with a press that pushed the spikes into the wood. One joint in [redacted] was found in which the bracket had not been pressed into the wood and thus was corrected prior to loading the roof. Numerous trusses were found to be out of vertical alignment and also to be supported by the outside walls only. Some trusses had as much as 1/2-inch clearance between the bottom chord and the interior walls of the house.

The roof slope was 1 to 3, typical of the other houses in the area. By draping bed sheets over the edge of the house and placing soil along the edge of the sheet on the roof, a soil roll was made (Figure 2.23) to hold the soil on the slope of the roof. The overhanging portion of the sheet was folded over the soil and up on the roof; soil could then be placed on the roof.

The roof on the front side of the house could be covered by shoveling soil out of the bucket on the front-end loader as shown in Figure 2.24. A bucket brigade was formed from the front to the back of the roof to cover the backside. The garage roof was also covered with the bucket brigade working from the ground to the roof. The roof could be reached easily by a man standing on the soil berm next to the walls of the house and also from the bed of the dump truck used to haul soil to the site. Time required to place the soil on the roof of the house by the various methods used is given in Table 2.1.

Deflection measurements made at the quarter points on selected roof trusses showed an average roof deflection of 3/8 inch with 4 inches of soil cover. Most of this deflection was due to the settlement of the trusses on the interior partitions of the house. On the structural test section of the roof, the soil depth was increased to 12 inches. This section of the roof deflected 4/8 inch and an additional 1/8 inch after supporting the soil load overnight, for a total of 5/8-inch deflection from the no load position. Some of the 2 by 4 braces in the trusses bowed slightly under the 12-inch soil loading; this was probably caused by poor vertical alignment of the roof trusses. The soil depth was increased to 18 inches over the test section of the roof without any

increase in deflection. In a misty rain on Friday afternoon, the soil cover was increased to 24 inches on the frontside of the roof and left at 18 inches on the backside. The rains of Friday amounted to 0.3 inch, which only dampened the loose soil on the roof of the house. Approximately 1 inch of rain fell on Sunday. Monday morning the portion of the roof covered with 4 inches of soil was covered with 4 inches of mud. On the structural test section of the roof where the soil depth was 18 and 24 inches, the soil had settled approximately 2 inches and was sticky for the top 2 or 3 inches.

With the weight of the wet soil for several days, the deflection increased to 15/16 inch. Figure 2.25 shows the house after the rain. Water seeped under the soil roll around the edge of the roof; however, none of the soil had washed off. When the roof was inspected on Monday, the 2 by 4 braces at the center of the truss were bowing as much as 1-1/2 inches. Some of the bowed braces can be seen in Figure 2.26. When the house was inspected on Tuesday, one of the truss braces had broken (Figure 2.27) at the location of a knot. The roof in that area sagged approximately 2 inches, and the recorded deflection locations showed an average deflection of 13/16 inch. Apparently, when the roof sagged over the broken truss, it bowed up slightly in the areas in which deflections were being recorded.

From soil density measurements, the loading on the frontside and backside of the roof in the structural test area was 180 and 130 psf, respectively. Since one of the braces in a truss had broken under this loading, no additional soil was added to the roof. The 3/8-inch-thick plywood sheathing had not bowed sufficiently between the trusses to obtain reliable deflection measurements. At most, the plywood had sagged 1/8 to 1/4 inch between the trusses.

The soil remained on the roof of the house for several more days with no further damage while plans were being completed for removing the soil.

2.3.3 Restoration of the House. The soil roll around the edge of the roof was removed and the soil on the roof was raked off in 48 man-hours. Soil piled against the exterior walls and on the roof of the

house was removed with a front-end loader in 4 hours and dumped in the eroded area of the backyard. Hand removal would have required almost as much effort as the original placement.

Figure 2.28 shows the house with the soil removed, and a closeup of the brickwork on the end of the house is shown in Figure 2.29. Due to limited space at the ends of the house, the garage roof and walls were torn down to allow the front-end loader access to the backyard where the soil was dumped. The entire house including the concrete slab foundation had to be removed from the lot upon completion of the test. The only damage to the house from the soil loading was the broken brace in one of the roof trusses. Soil remaining on the brickwork could be swept off with a stiff broom. Soil remaining on the roof had to be washed off. The interior doors used to cover the two front windows were undamaged and reusable.

Had the interior ceiling not been removed, the deflection of the roof trusses would have caused cracks in the joint between the ceilings and the walls. Drywall joint compound and painting are all that would be necessary to repair these cracks.

The interior garage wall that had soil piled against it needed cleaning and repainting but was undamaged by the loading and moisture in the soil.

The entire house lot needed grading and seeding to put the house site back into its original condition. Had the shrubbery from around the house been removed carefully it could have been replanted in its original location.

2.3.4 Test Results and Conclusions. Allowing 10 square feet per person, the shelter as constructed would have space for 80 persons. Upgrading the house required the placing of 170 yd^3 of soil on the roof and against the sides of the house. With 80 people occupying the shelter, each one would have to move 2-1/8 yd^3 of soil to construct the shelter. The effort and time required to construct the shelter can be reduced substantially if the soil is piled against the walls of the shelter with equipment such as a front-end loader. Placing soil on the roof of a structure is a much more difficult problem. The soil was

lifted to the house roof with a front-end loader where it was spread by the workmen. This proved to be an inefficient use of the front-end loader. If soil is placed on the roof of a structure with machinery a conveyor belt or mechanical lift system should be used. Cranes equipped with clamshell buckets should not be used because the impact loading on the roof from soil dropped from the bucket could cause the roof system to fail.

Neither the exterior brick veneer walls nor the interior garage wall was damaged by the soil. The exterior walls of commercial or public buildings, unless they are glass-curtain or corrugated metal walls, will be as strong or stronger than the wall of the house that was upgraded. Therefore, the only structural problem expected to occur when the walls of commercial or public buildings are upgraded is covering of the openings. The windows of the house were successfully covered with the interior doors or plywood sheets. If the openings are large, it will be necessary to build a structural framework that can support the plywood or interior door covering.

The prefabricated truss roof system of the test house supported up to 24 inches of soil with only minor damage to one of the trusses. The trusses had no trouble supporting 12 inches of soil cover, which was the depth necessary to obtain a PF greater than 40 for the shelter. An analysis of a conventionally framed roof system having the same span and slope as that of the test structure and designed by Federal Housing Administration (FHA) specifications (Reference 2) showed that the onsite built roof would also support 12 inches of soil without damage. Deflections of the roof trusses will cause some cracking at the junction of the interior walls and ceiling. This can be repaired with drywall joint compound and repainting.

A soil roll along the lower edge of the roof held the soil in place during placement and rains that occurred during the construction. The soil roll allowed the rainwater to drain from the roof without washing away the soil. Although the soil roll barrier has not been tested on steeper slope, it is expected to work satisfactorily on all roofs normally used for house construction.

TABLE 2.1 EFFORT AND SOIL PLACEMENT RATES FOR UPGRADING DEMONSTRATION

Soil Placement Method	Area Covered	Quantity of Soil Placed yd^3	Time Required Man-hours	Placement Rate yd^3/hr
Front-end loader	Front wall	33	0.75	44.0
Wheelbarrow and shovels	Garage wall, 3 feet high	8-1/4	6.6	1.25
	Garage wall, 3 to 4 feet high	14-1/4	15.0	0.95
	Garage wall, 4 to 6 feet high	3-3/4	10.5	0.36
	Garage wall, total	26-1/4	32.1	0.82
Bucket brigade and front-end loader	Roof	10	15.0	0.67
Bucket brigade from truck bed	Roof	2-3/4	10.0	0.28
Bucket brigade from ground	Roof	3-3/4	20.0	0.19
Front-end loader	All exterior walls	136	3.25*	44.0
Hand labor	All exterior walls	136	166.0*	0.82
Bucket brigade and front-end loader	Entire roof	56	84.0*	0.67
Bucket brigade from truck bed	Entire roof	56	200.0*	0.28
Bucket brigade from ground	Entire roof	56	295.0*	0.19

* Estimated using values shown for placement rate.

Figure 2.1 Aerial view of test house location.

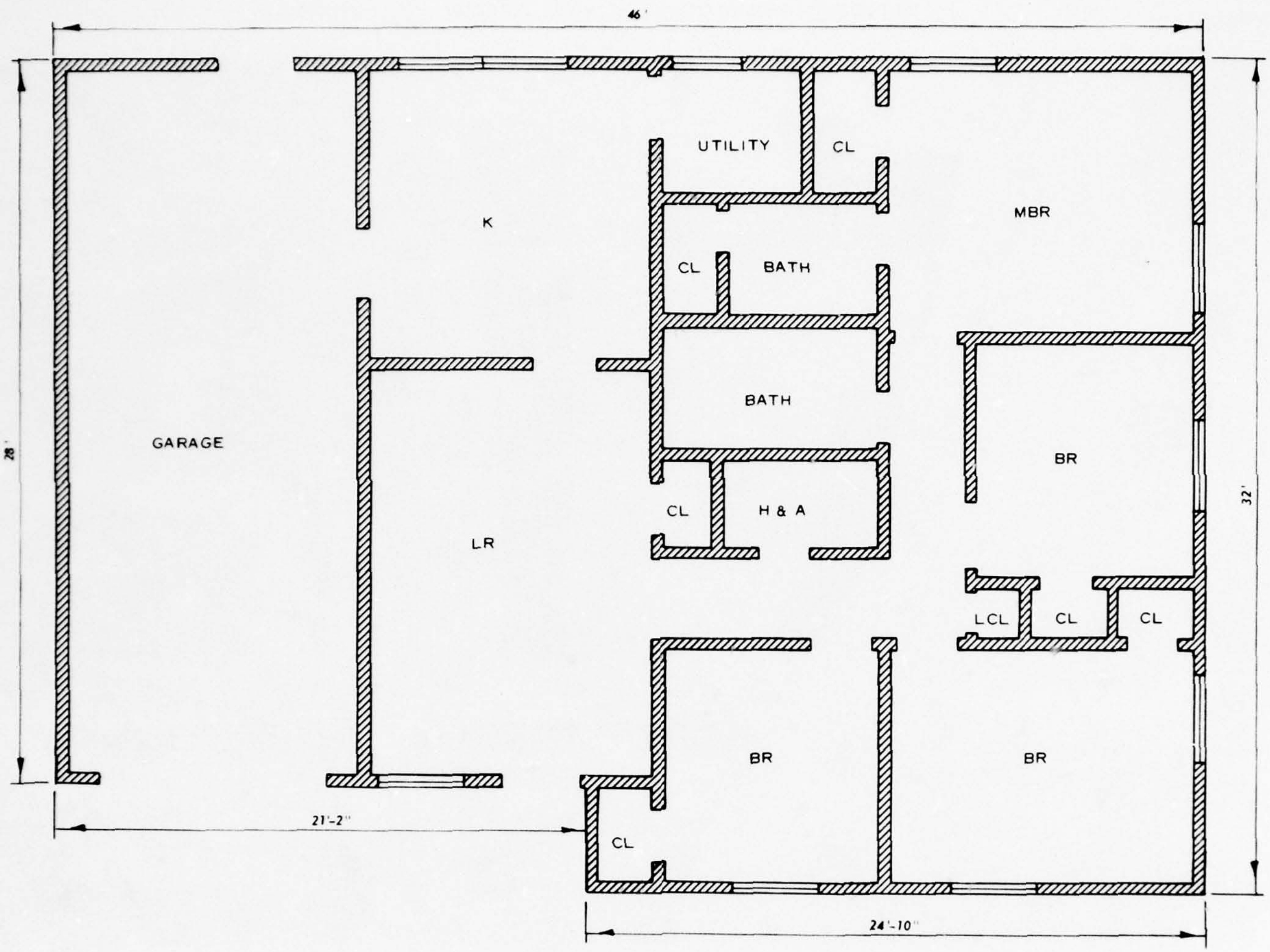


Figure 2.2 Floor plan of test house.

a. Front view.

b. Rear view.

Figure 2.3 Views of test house.

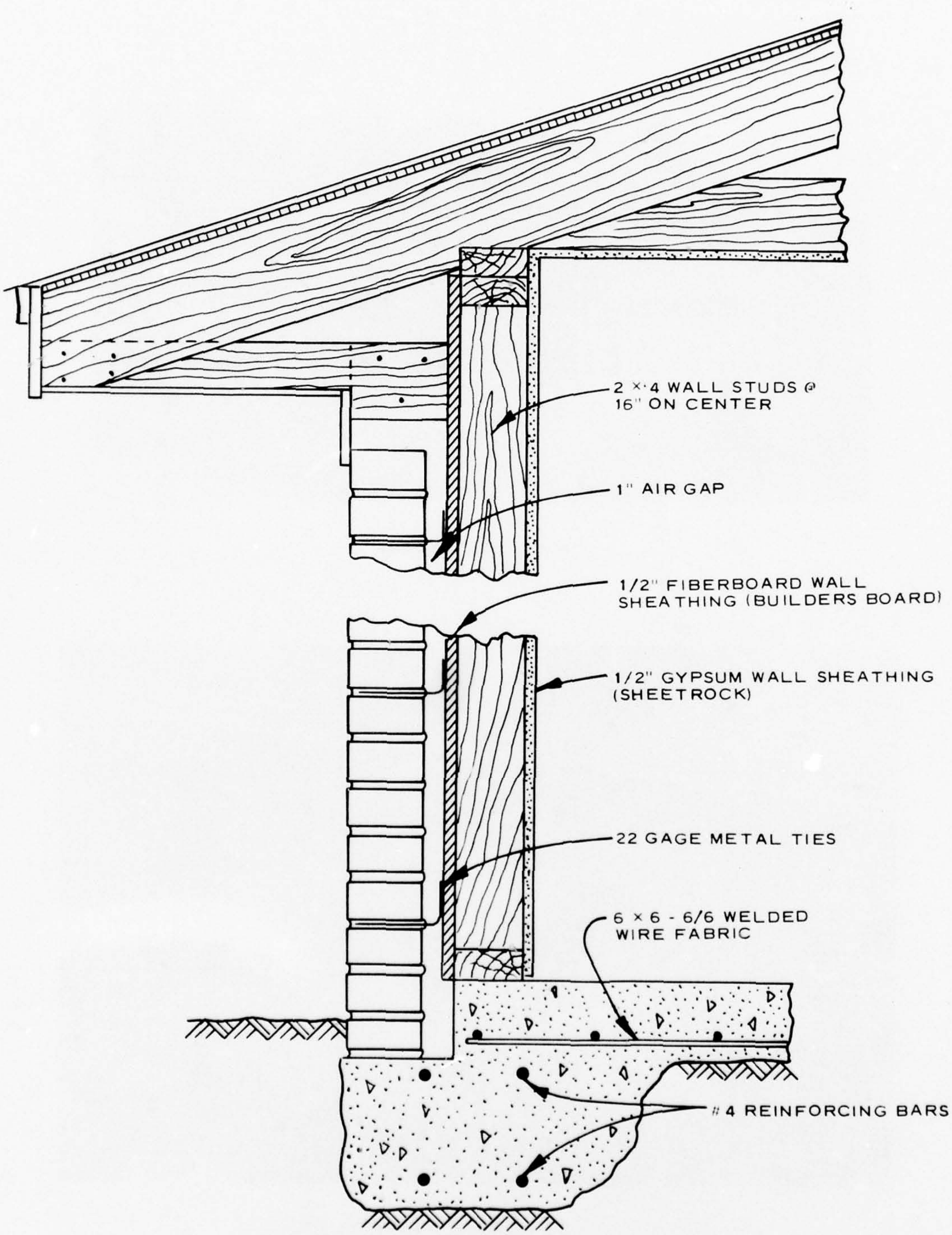
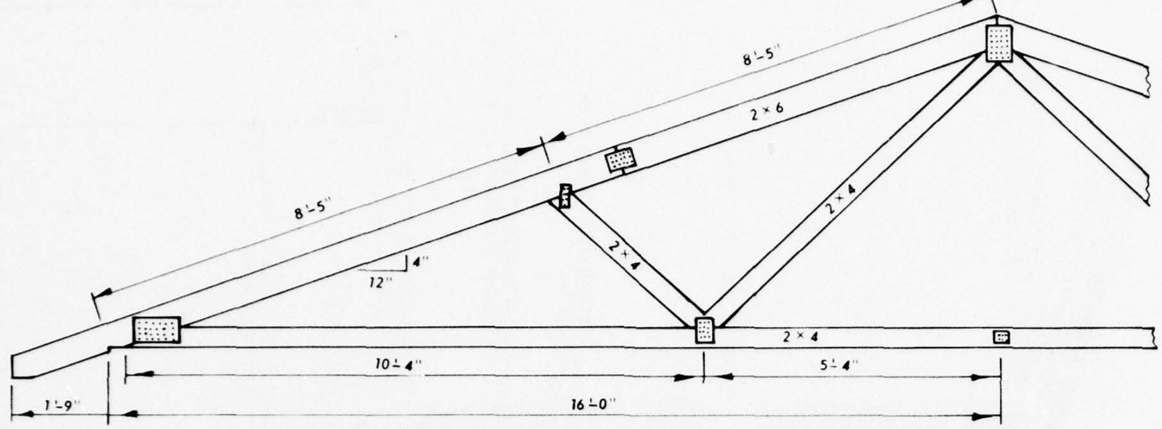
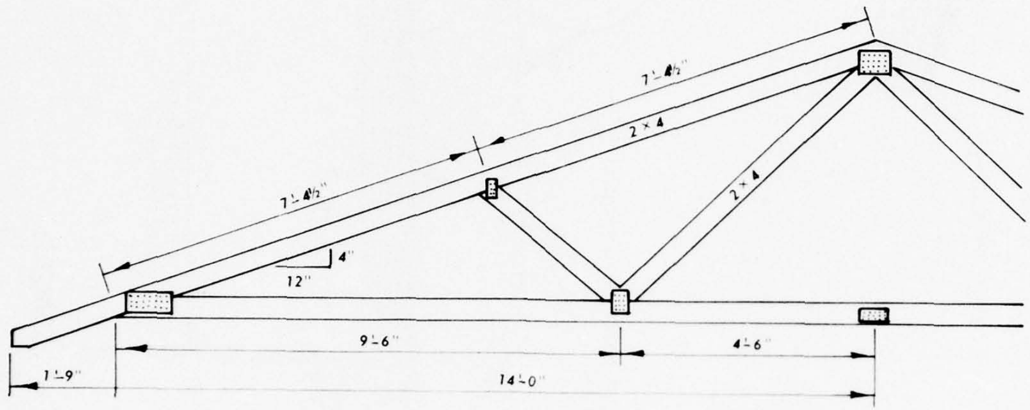


Figure 2.4 Exterior wall cross section.



a. Truss over main portion of house.



b. Truss over garage.

Figure 2.5 Prefrabricated roof truss.

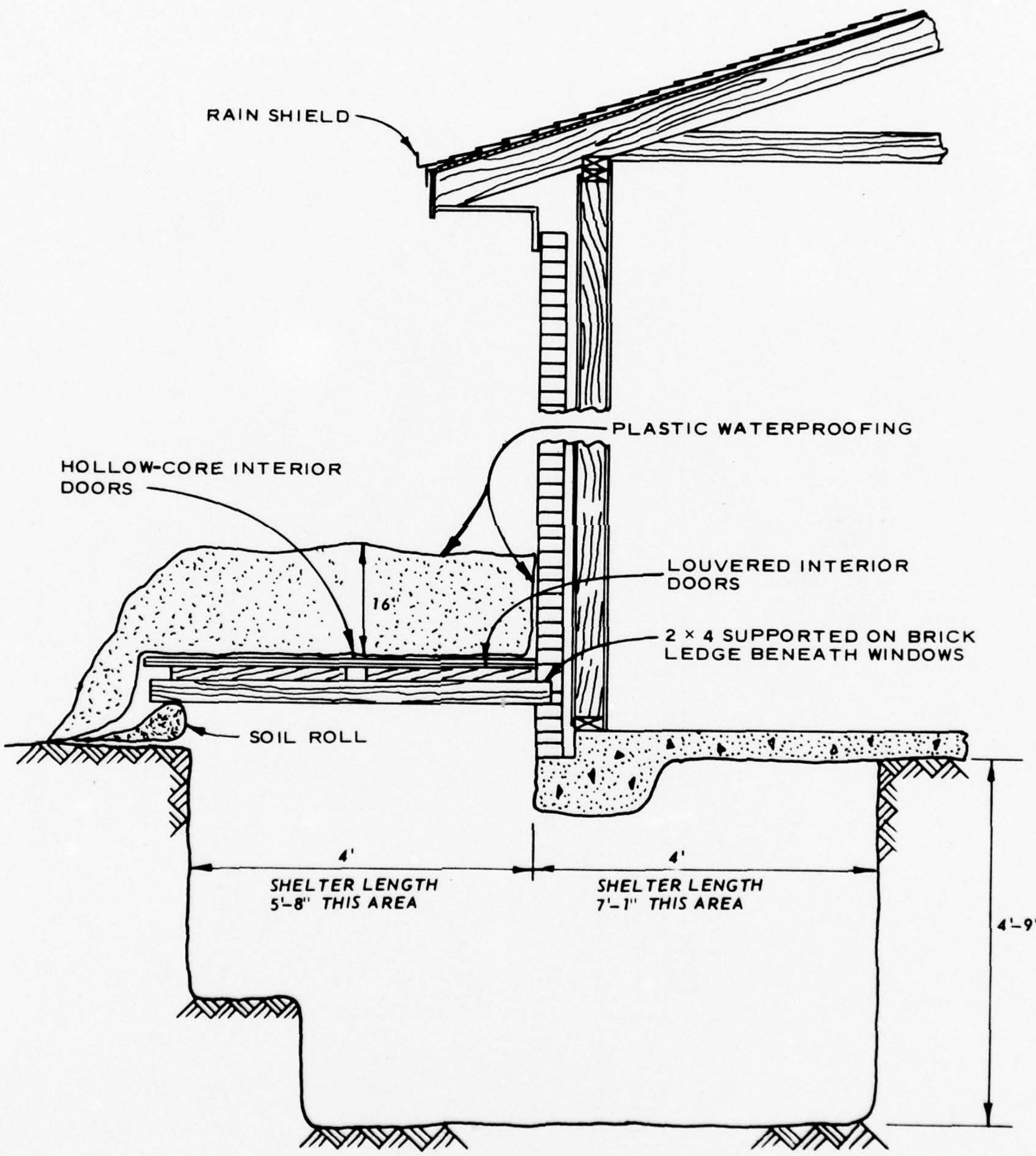
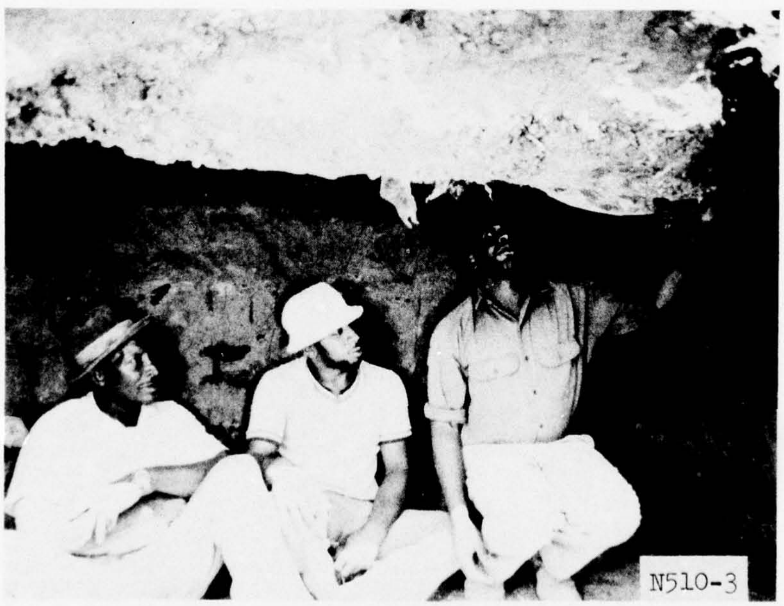


Figure 2.6 Cross section of single-family shelter.

a. Access trench.



b. Shelter under slab.

Figure 2.7 Completed single-family shelter.

Figure 2.8 Hollow-core doors being used to cover access trench.

Figure 2.9 Waterproofing the access trench.

Figure 2.10 Shelter closure.

Figure 2.11 Shelter interior with entrance open.

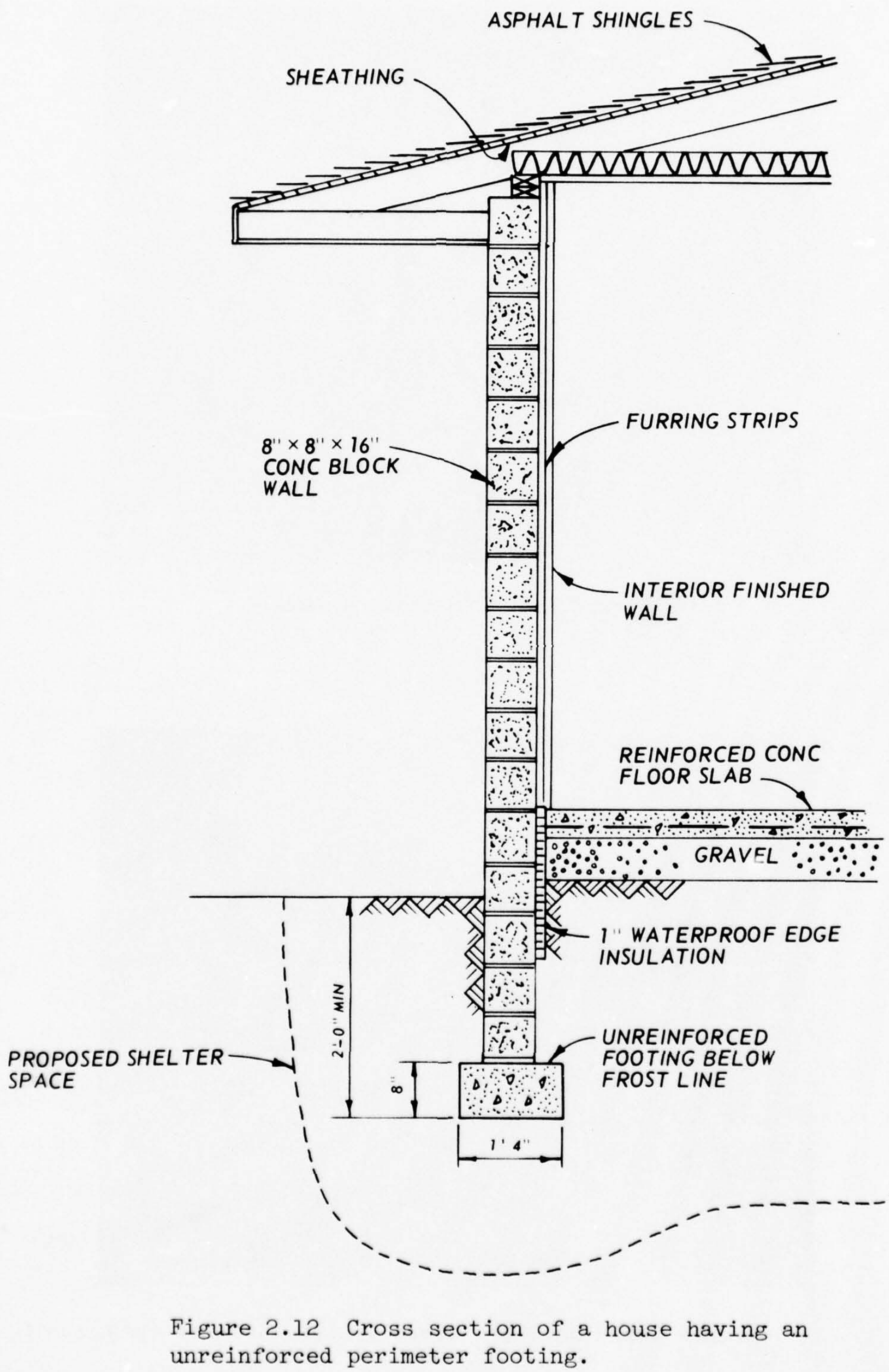


Figure 2.12 Cross section of a house having an unreinforced perimeter footing.

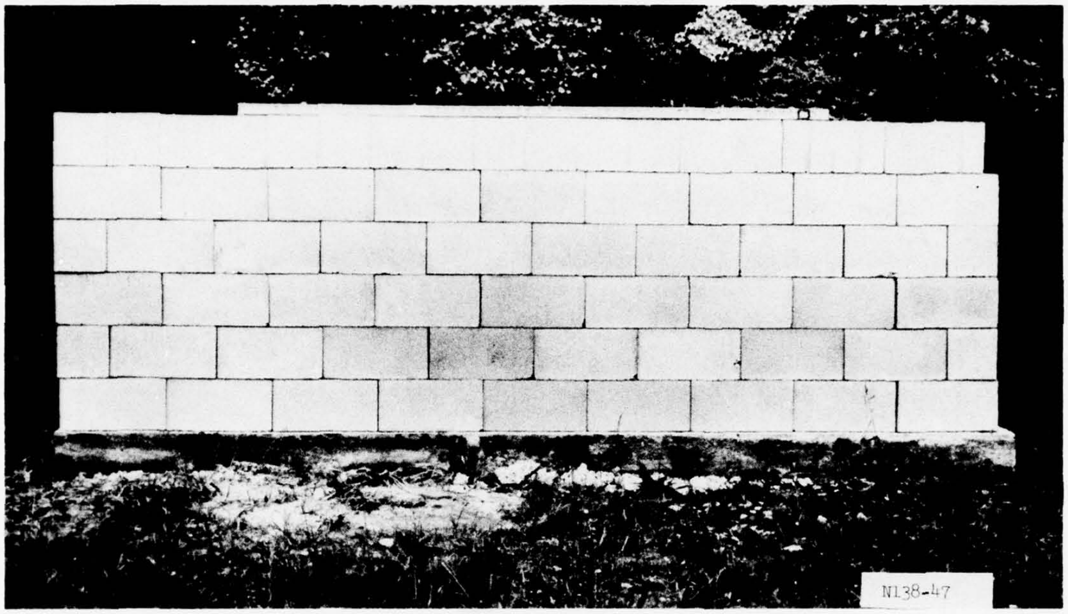


Figure 2.13 Completed test footing and concrete block wall.

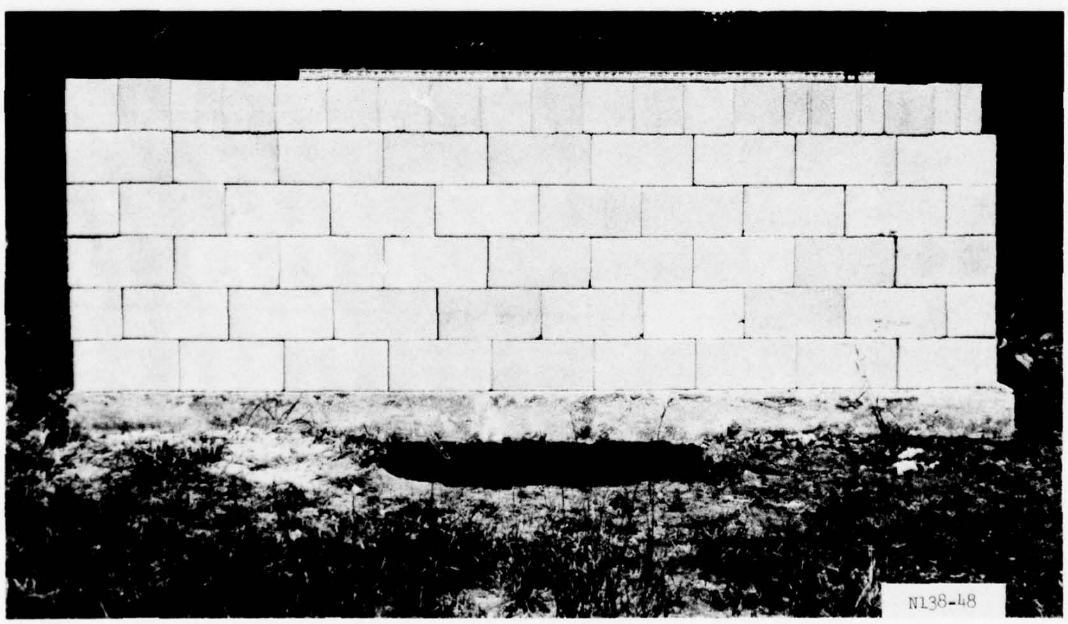
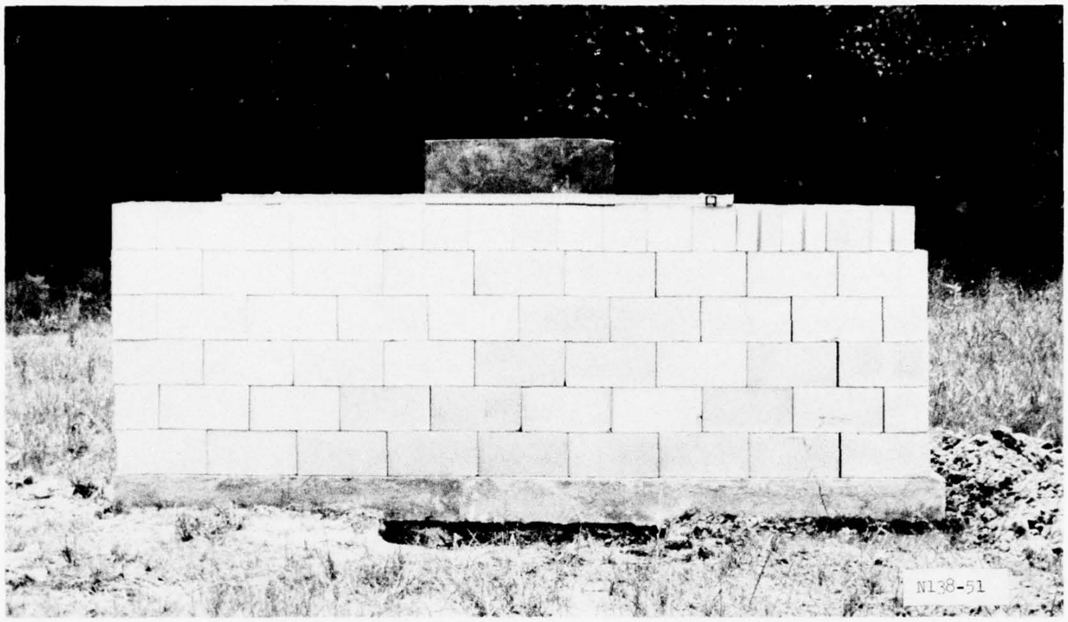
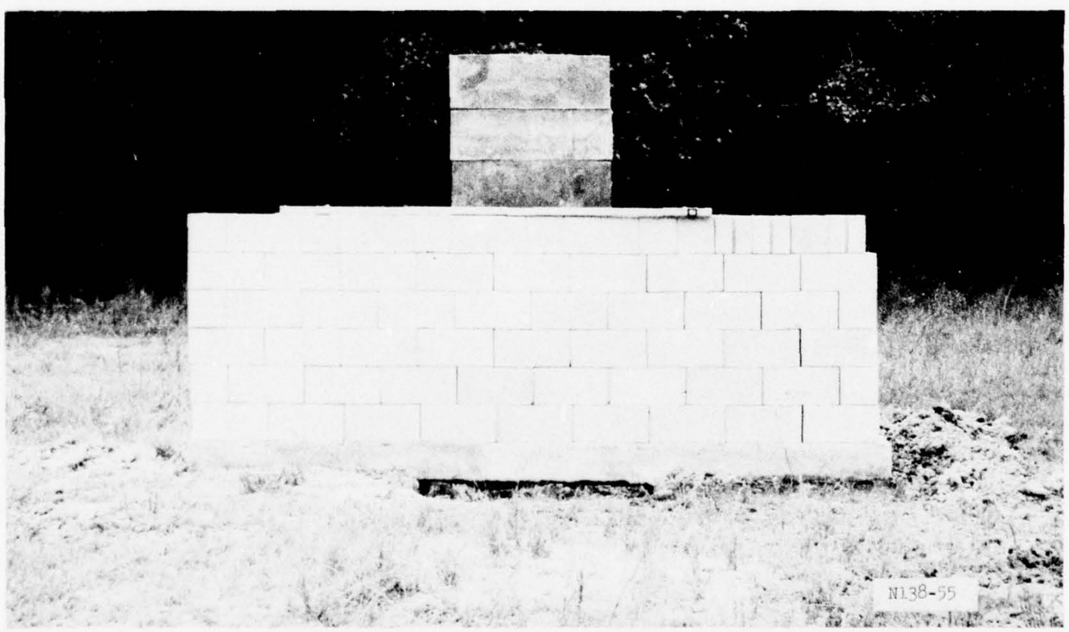


Figure 2.14 Unreinforced footing spanning a 4-foot trench.



a. 1000-pound added weight.



b. 3000-pound added weight.

Figure 2.15 Unreinforced footing supporting added weights.

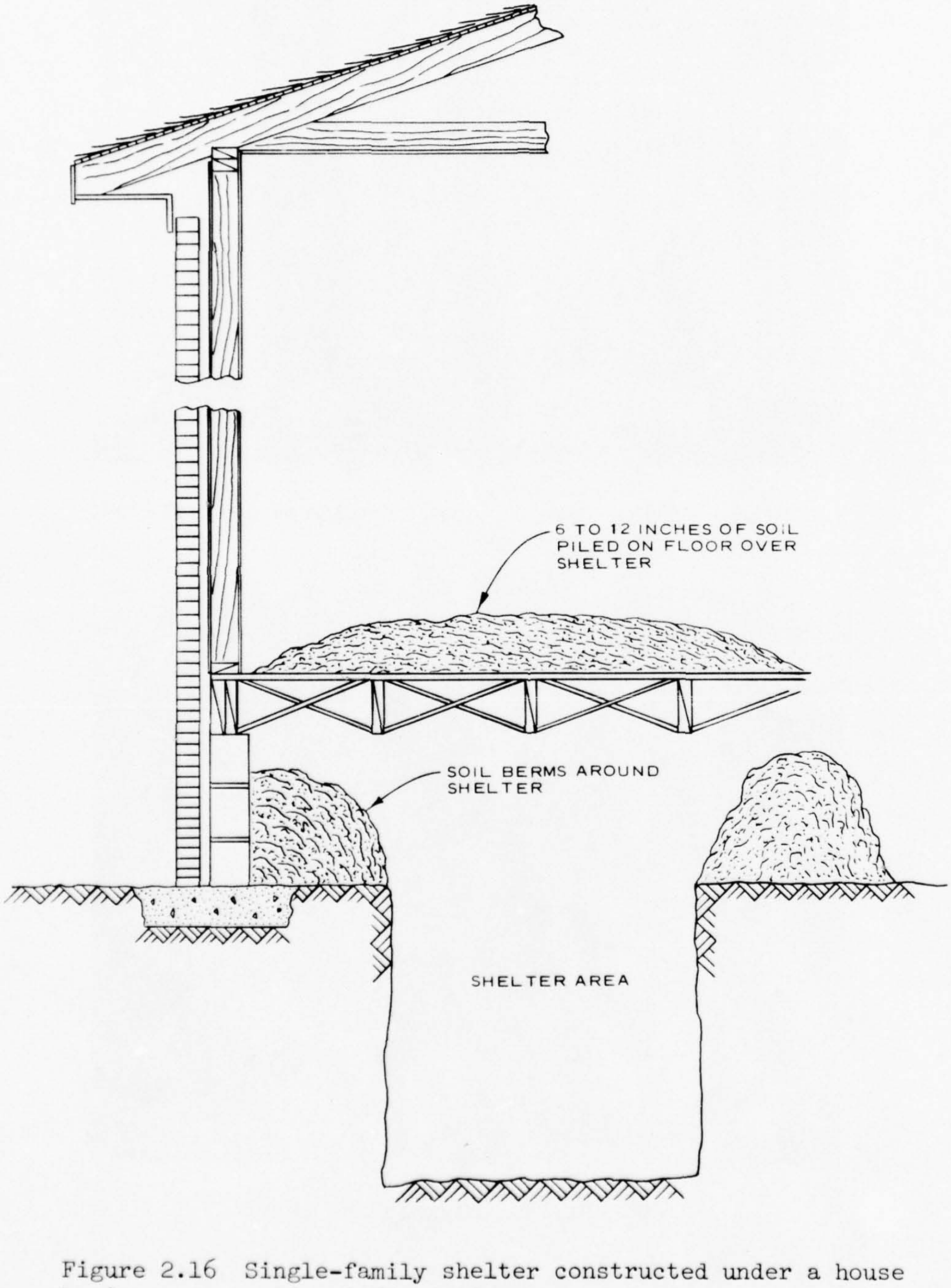
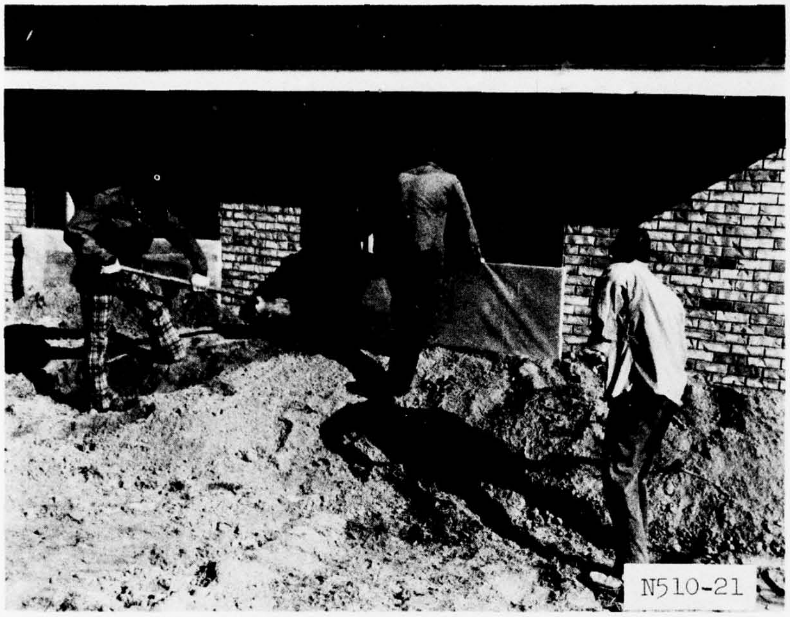


Figure 2.16 Single-family shelter constructed under a house having a conventional foundation.

a. Placing 4- by 8-foot sheet of plywood over window.

b. Nailing plywood in place.

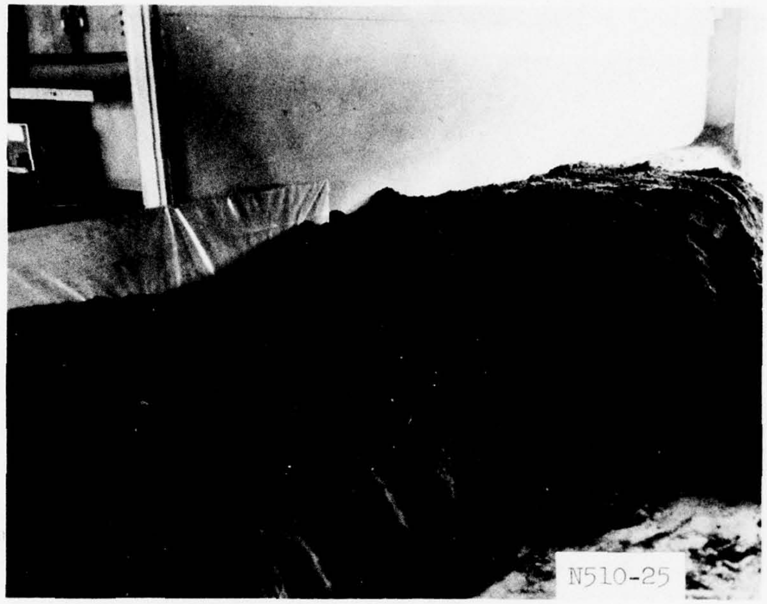
Figure 2.17 Plywood being placed over the front window of the house.



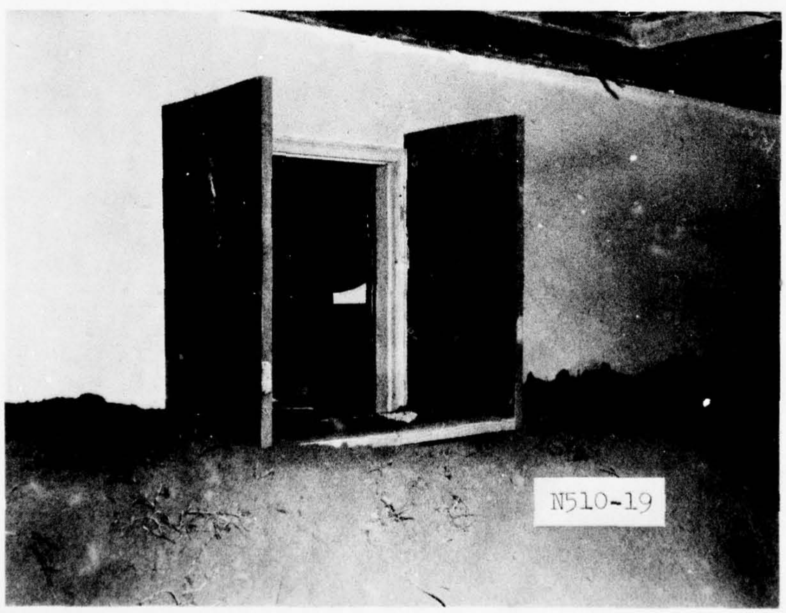
a. First level of doors being placed.

b. Second level of doors being placed.

Figure 2.18 Covering windows with interior doors.



a. Covering of lower half of kitchen door to make shelter entrance.



b. Placement of wing walls beside shelter entrance.

Figure 2.19 Fabrication of shelter entrance

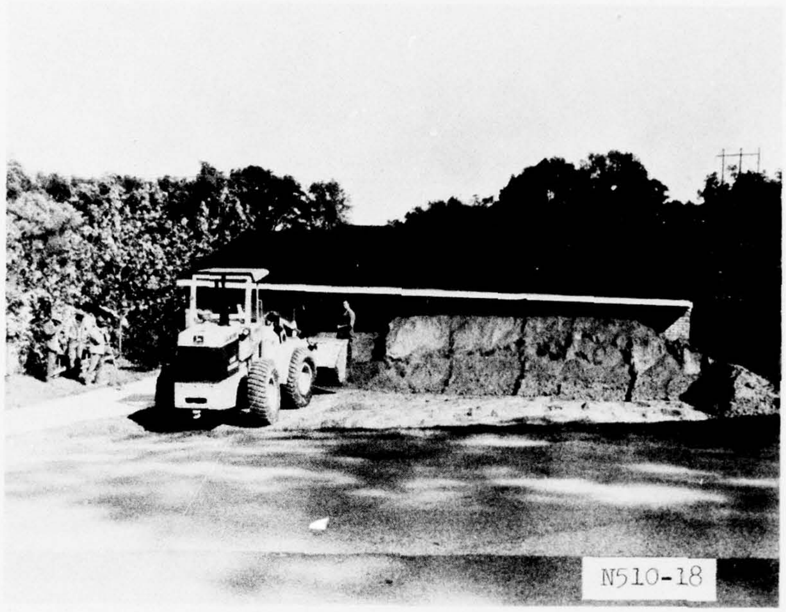


Figure 2.20 Soil placement with a front-end loader.

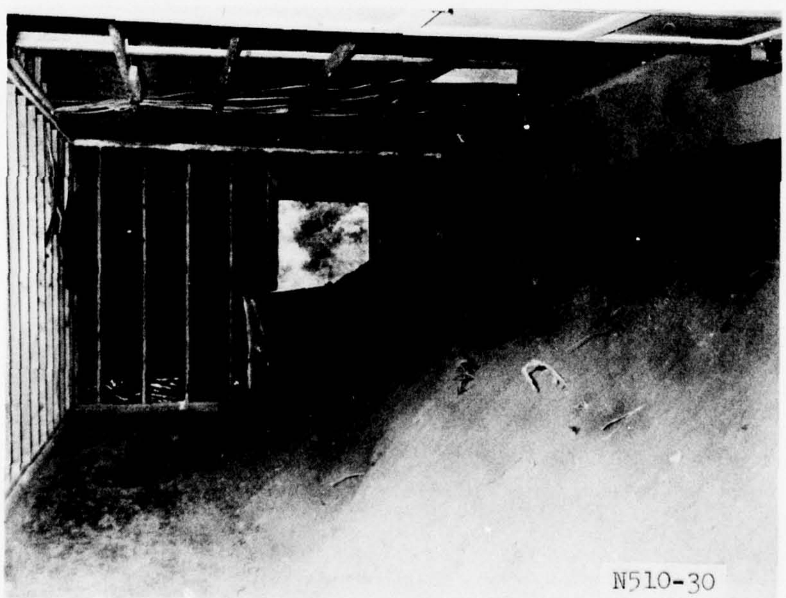


Figure 2.21 Completed garage wall.

Figure 2.22 Soil being placed with a crane.

Figure 2.23 Fabrication of soil roll along edge of roof.

Figure 2.24 Front-end loader being used to lift soil to house roof.

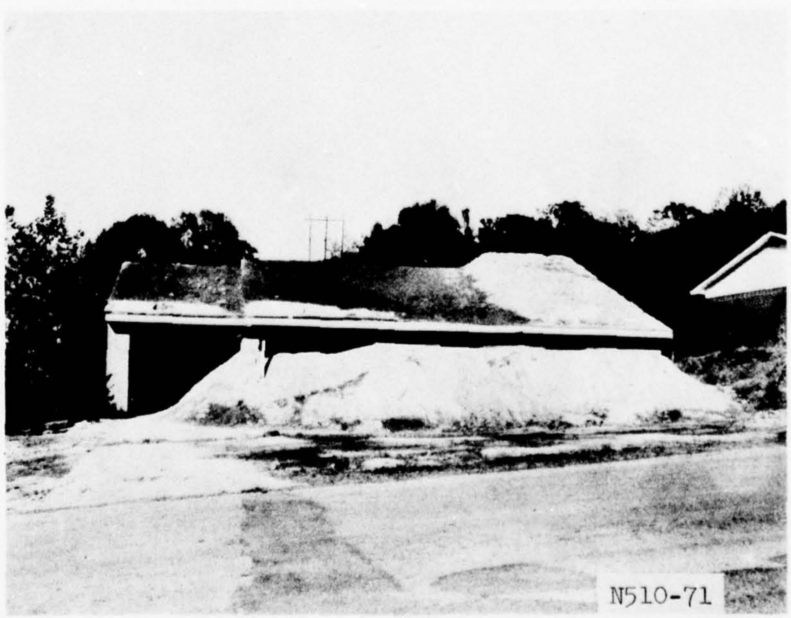


Figure 2.25 Completed shelter after weekend rain.

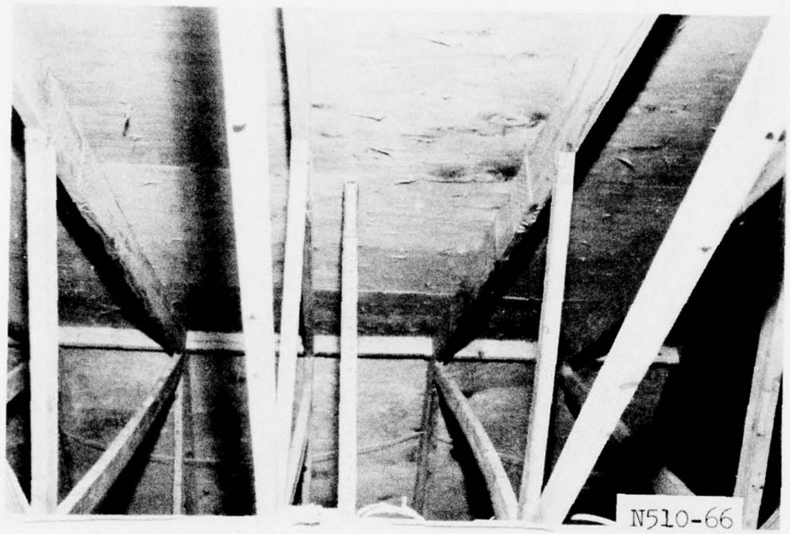


Figure 2.26 Bowed truss members.

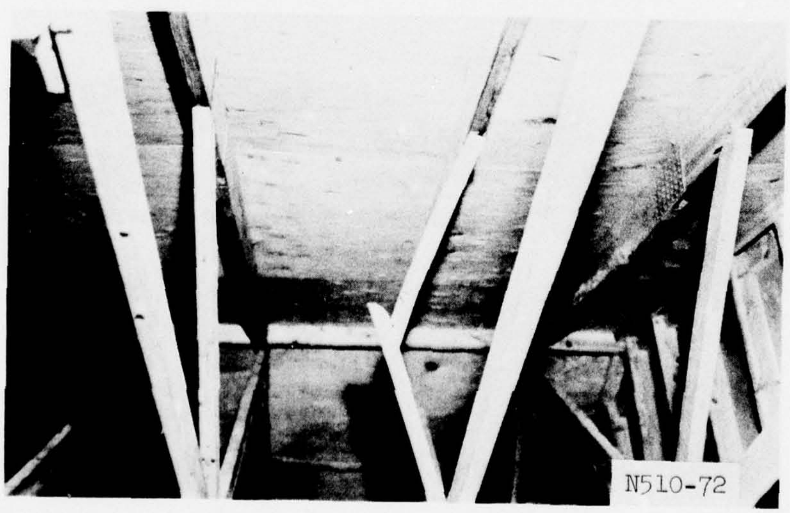


Figure 2.27 Broken truss member.

Figure 2.28 House with soil cover removed.

Figure 2.29 Closeup of brick veneer wall with soil removed.

CHAPTER 3

LABORATORY TEST AND ANALYSIS OF SELECTED ROOF SYSTEMS

The design live load for flat roofs or roofs with slopes less than 4 in 12 is 12 to 20 psf depending on the tributary loaded area for the structural members supporting the roof. Roofs that are used for terraces or some special purpose or are subjected to snow loads are designed accordingly. The dead load consists of a waterproofing layer, insulating layer, support deck, and roof framing. The design dead load varies about 5 to 25 psf depending on the type of roof system. Adding a soil layer 12 inches thick for radiation protection adds approximately 100 psf to the load that the roof must carry. The safety factors built into the design of a wood or concrete roof system to account for strength variations, nonumiform loading, assumptions and simplifications inherent in the analysis, and variation in structural behavior from the assumed behavior may be sufficient for some roof systems to carry safely the added soil loading without modifying the roof support system.

In single-story construction most roof support systems consist of one or a combination of the following: steel open-web joists (OWJ), steel beams, concrete beams, steel trusses, steel space frames, wood joists, and wood trusses. The roof decks consist of concrete slabs in place or precast; patent steel decking alone or with insulating concrete or rigid insulating board; corrugated metal, fiberglass, or cement asbestos board; and plywood. In some cases, such as the corrugated metal roof, the decking is also the waterproofing layer for the roof. Other roof decks require a separate waterproofing layer such as asphalt shingles or a built-up felt and gravel waterproofing layer. The insulating and waterproofing layers do not provide any of the load-carrying capacity of the roof system; therefore, they were not included in the laboratory test on roof systems.

Two types of roofs were tested in the laboratory: the steel OWJ with steel decking and the wood joist-plywood roof. Both roof systems were patterned after roofs located on single-story buildings in the Vicksburg, Mississippi, area. The OWJ roof tested was patterned after

the roof on a local school and, according to the architect, was similar to that used in modern school construction in most parts of the country. The laboratory tests were conducted to evaluate the overload capacity of the two roof systems and to determine methods to increase the load-carrying capacity. Analyses of several other types of roofs were made utilizing data from previous laboratory tests.

3.1 OWJ ROOF SYSTEMS

3.1.1 Description. A typical section of an OWJ roof system is shown in Figure 3.1. The roof over the hallways and classroom areas of a local vocational school provided the design for the two OWJ roof sections constructed for static-load testing. OWJ's at 4 feet on center provided support for the roof in both areas of the school. The hallways and classrooms were spanned by 10-foot-long 8J3 and 28-foot-long 18J6 joists, respectively.

Plans for the two roof sections tested are shown in Figure 3.2. The roof sections were 12 feet wide and consisted of 4 OWJ's at 4 feet on center with metal roof decking spot-welded to the top chord of the joist at 24 inches on center. The OWJ's were purchased from the same manufacturer as the joists for the school roof. The steel roof decking was not obtained from the supplier of the material for the school roof. The corrugations were slightly different; however, the mechanical properties of the two decking materials were similar. The completed test roof constructed using 10-foot OWJ is shown in Figure 3.3.

3.1.2 Test Procedures. For the static load test the roof sections were supported at the ends by a 3-foot-high wooden frame (Figure 3.3). The school's roof was supported by concrete block walls and secured with anchor bolts. The test roof sections were not fastened to the wood frame supports in order to determine if the joists would slip off the supports when undergoing large deflections or fail by buckling first.

The roof sections were statically loaded by placing sand on the roof with an overhead crane and a clamshell bucket. Sand was placed in 4-inch-deep increments and held on the roof by a wall of sandbags placed around its perimeter. The sand used for the loading is known locally as

Cook's Bayou and is normally used for buried model tests conducted in the Large Blast Load Generator (LBLG) facility operated by the Structures Division of WEL. The fine, fairly uniform sand has a minimum and maximum dry unit weight of 93.3 and 110.3 pcf, respectively. As placed, the sand density was approximately 100 pcf.

Six static load tests were conducted on the two OWJ roof systems. The roofs were loaded twice each with the joists supported at the ends only. The 28-foot OWJ roof was loaded once with an added support at midspan and once with added supports near the one-third points.

At each loading increment, deflection measurements were obtained by using a surveyor's level to read rulers taped to the joists. Deflection measurement locations were at midspan for each joist when the joists were supported at the ends only and when one-third point supports were added. Deflection measurements were made at or near the one-third point when a midspan support was added.

3.1.3 Results and Analysis of the 10-Foot OWJ Roof Test. The 10-foot-span OWJ roof system was load tested twice. The first load test was conducted to obtain load-deflection data for comparison with analysis and for predictions of the load-deflection curve for the 28-foot-span OWJ roof system. The second load test was conducted to obtain failure modes and loading for the roof system. Previous tests on OWJ roofs (Reference 3) indicated that the failure load was approximately 1.8 times the allowable load from standard joist load tables (Reference 4). The 8J3 joist had an allowable load of 400 pounds per linear foot (plf) which, according to the test in Reference 3, would give a failure load of 720 plf or 21-1/2 inches of sand. The first loading was stopped at 20 inches of sand. The load-deflection curve for this loading is shown in Figure 3.4. Due to the surveyor's level being moved, the last deflection measurement recorded was with 16 inches of sand on the roof. As can be seen in Figure 3.5, the roof system supported 20 inches of sand without any signs of distress. The end view of the OWJ in Figure 3.5b shows a slight bowing of the metal roof deck between the joists. No damage to the joist nor the metal decking was found when the sand was removed.

After completing the test of the 28-foot-long OWJ roof system, the 10-foot OWJ roof was retested to failure. Figure 3.6 shows the roof system supporting 36 inches of sand, the maximum sand depth obtained. The load-deflection curve for this loading is shown in Figure 3.4. The midspan deflection reached a maximum value of 7/8 inch, which was twice the maximum live load deflection of L/240 allowed by most building codes. With 36 inches of sand on the roof, the two interior joists were supporting three times their allowed uniform loading from the joist load tables, and the end reactions were 1.75 times the maximum given in the load tables. At the time the photographs were made for Figure 3.6 the joists were still showing no signs of failure. However, note the edge of the metal decking in Figure 3.6b. The decking had deflected 2 inches at midspan between the joists, and the corrugations were beginning to buckle where they crossed the joist.

Since the decking was failing rather than the joists, the test was stopped. As the sand was being removed for a closer inspection of the metal roof decking, the two center joists started to fail. Figure 3.7 shows the initiation of the failure at the second web member from the end of the joist. The two middle joists of the roof system are shown in Figure 3.8 after removal from the roof. The two outside roof joists were undamaged.

The two joists failed in a similar manner to those tested in Reference 3. A truss analysis was used to predict the midspan deflection and forces in the chord and web members of the OWJ. The predicted load-deflection curve for the joists is shown along with the test data in Figure 3.4. The truss analysis indicated that the joists were about twice as stiff as they actually were. Reference 3 obtained good correlation between calculated and experimental load-deflection data using the formula for the deflection of a uniformly loaded simply supported beam with a constant multiplier

$$\Delta = 1.15\ (5wL^4 0.384EI) \qquad (3.1)$$

where

Δ = midspan deflection, inches

w = uniform load, lb/in.

L = clear span, inches

E = Young's modulus, psi

I = moment of inertia, inches^4

The moment of inertia I for the joists was calculated using only the top and bottom chords of the joists. Using this formula the load-deflection curve was almost the same as that obtained with the truss analysis (Figure 3.4). The experimental load-deflection curve for both loadings becomes much stiffer at loads above 250 lb/ft, indicating that there was considerable slack in the roof system at the beginning of the test. The second slope in the experimental load-deflection curve, particularly the curve from the first loading, is nearly the same as that of the two computed curves.

From the truss analysis, the web member that failed was being subjected to an axial force of 8000 pounds. Assuming pinned connections, the calculated critical buckling load from the Euler formula was 4600 pounds. If the ends are assumed fixed, the critical buckling load is 18,500 pounds. Although the web members of the joists are continuous and welded at each intersection with the top and bottom chords, they are acting more as a pinned end member than a fixed end member.

Rotation of the ends of the joists causing slippage from the supports was expected to be a problem during large deflections. However, no slippage at the supports was observed in this test even with the ends not secured.

3.1.4 Results and Analysis of the 28-Foot OWJ Roof Test. Four load tests were conducted on the 28-foot OWJ roof system. The first of these was conducted with the ends supported only to obtain load-deflection data that could be compared with the analysis. The second and third tests were conducted to evaluate the addition of midspan and one-third point supports, respectively. The final load test carried the roof system to failure with supports at the ends only in order to determine collapse loads and mechanisms. The following is a discussion of the results of the load tests.

1. End supports. Figure 3.9 shows the roof system supporting 12 inches of sand, which was the maximum load for the first test. Close correlation was obtained between the predicted load-deflection curve using a truss analysis or the modified beam formula, Equation 3.1, and the experimental load-deflection curve. After the two tests with added interior supports, the roof system was reloaded with end supports only. Approximately 16 inches of sand was placed on the roof and left for 5 days. Deflection measurements taken at the end of the first and fifth day recorded an increase in deflection due to creep of 0.10 and 0.39 inch, respectively. Loading was continued at the end of 5 days in 4-inch increments as before. As the last clamshell bucket of sand was being placed for the 20-inch sand depth, the roof system collapsed. Figure 3.10 shows the roof immediately after collapse and after the sand and metal decking had been removed. Failure was apparently initiated by buckling of the top chord of one of the middle two joints. The two outside joists were then able to rotate away from the loading and fall off the ends of the support frame. The individual joists are shown in Figure 3.11, and closeups of the buckled top chord of the two interior joists are shown in Figure 3.12. Note in Figure 3.11 the two outside joists, Nos. 1 and 4, received almost no damage. Also, the failure of the two interior joists occurred due to the buckling of the top chord at midspan rather than buckling of the web members near the ends as in the 10-foot-span OWJ roof and in the joists that were tested in Reference 3.

The load-deflection curve for the 28-foot interior joist is shown in Figure 3.13 along with the load-deflection curves obtained from the truss analysis and the modified beam formula. Four inches of sand on the roof represents the approximate weight of the insulating concrete and built-up roofing on the prototype roof. Therefore, the roof system, when supporting the 16 inches of sand, was equivalent to the school roof being loaded with 12 inches of soil, the desired loading for adequate radiation protection. The maximum initial deflection with 16 inches of sand on the roof was 1.9 inches. This deflection is equal to 1/168 of the clear span. The maximum deflection allowed by most building codes

is 1/240 of the clear span. From the load tables for OWJ joists, the load required to obtain a deflection of 1/240 of the clear span is 354 plf. This load level produced a midspan deflection of approximately 1.3 inches, which is 1/246 of the clear span, indicating that the tested joists were responding normally. The failure load is too close to the required load capacity for the roof to safely carry without additional supports for the soil cover necessary for radiation protection (failure load/required load for radiation protection = 650/515 = 1.26).

2. End and midspan supports. Supports were added near midspan for the second load test of the 28-foot joist. The added support was placed 1 foot from the center of the span in order for the support to be located at a joint between the joist's top chord and web. Two by four columns on each side of the joist supported a 2 by 4 beam on which the flanges of the top chord of the joist were supported (Figure 3.14a). Loading of the roof section was accomplished as in the previous test. During loading, deflections were monitored near the quarter points of the two middle joists. These deflections were several times those determined by analysis. The weld between the top chord and the web member was broken when the joists were inspected after loading with 12 inches of sand. The broken weld allowed the web member to rest directly on the 2 by 4 beam support. The round web member started crushing the edge of the 2 by 4 (Figure 3.14b). Adding the support had caused the joist to fail at a load lower than it had previously carried with end supports only.

The load-deflection curve for the deflections measured near the quarter points is shown in Figure 3.15. The curves are shown dashed from zero load and deflection to the first point at which measurements were made. The curve is nonlinear from the first measurement indicating that the weld was cracking from the initial loading. The beam and truss analysis predicted the system to be much stiffer than it was. Poor correlation between analysis and test results was due to the failure of the welded joint.

A midspan support carries approximately 63 percent of the uniform load on the roof system while each of the end supports carries only

18.5 percent of the load. With midspan supports, most of the load is being carried at a point on the joist that is not designed as a support. The manner in which the joist was supported and the midspan support carrying most of the load caused the weld failure.

3. End and one-third point supports. Supports were added near the one-third points of the span to reduce the loading from that carried by a midspan support. The two added supports carried 36.7 percent of each of the uniform load on the joist and the end supports each carried 13.3 percent. Moving the support from midspan reduced the load on the support by approximately 43 percent. A mockup of the support system with a 24-inch sand loading is shown in Figure 3.16a. The same 2 by 4 column and beam arrangement used for the midspan support was used for the one-third point supports. However, this time the round bar of the web rested on the 2 by 4 beam rather than supporting the joist by the flanges of the top chord. This relieved the shear on the weld between the top chord and the web bar. Under load the web bar cut into the 2 by 4 beam allowing the joist to deflect slightly at the supports (Figure 3.16b).

The roof system with the one-third point supports carried 24 inches of sand or a loading of approximately 200 psf. The two center joists were carrying a uniform load of 785 plf, which was 2.85 times the allowable uniform load from the joist load tables (Reference 4). The load-deflection curve for the middle joist is shown in Figure 3.17. The joist sat for 12 days with 16 inches of sand on the roof. During that time, the deflection increased approximately 20 percent due to creep. On continued loading the slope of the load-deflection curve increased indicating that the system was stiffer than during the initial loading. The beam and truss analysis again predicted the system to be much stiffer than it was. Crushing of the 2 by 4 support beam under the web members was not considered in the analysis.

3.1.5 Support Column. In an actual structure the added column supports would be 10 to 14 feet long. Columns used as added supports for the test roofs have sufficient cross sectional area to carry the roof loading but are too slender to be safe against buckling. A column

that will safely carry the expected loading is shown in Figure 3.18. Three 1/4-scale models (Figure 3.19) of this column were fabricated and tested in a universal testing machine. The buckling load for the model columns was three times the scaled load that the prototype columns are to support. Columns fabricated as shown in Figure 3.18 can be used as one-third point supports for OWJ roofs having spans of up to 28 feet (the span of the test roof) and covered with 1 foot of soil. The number of columns can be reduced if they are located as shown in Figure 3.20. In this configuration the safety factor against buckling is reduced from 3 to 1.5; therefore, the columns should be braced. The beam spanning between the columns (Figure 3.20) needs to be equivalent to three 2 by 10's.

3.2 WOOD JOIST ROOF SYSTEM

<u>3.2.1 Description and Test Procedures</u>. The test roof was copied from one used on a local office building (Figure 3.21), which had a slope of 2/3 inch per foot. The roof was composed of 2- by 8-inch joists at 24 inches on center supporting 3/4-inch plywood decking and a built-up type roofing material. Maximum clear span of the roof was 12 feet over the office area of the building.

The roof section fabricated for static load tests was similar to one half of the office building roof. Plans for the test roof are shown in Figure 3.22 and the completed roof is shown in Figure 3.23. Decking for the test roof section was 3/4-inch-thick 4- by 8-foot sheets of A-C pine plywood and the joists were 2- by 8-inch Grade C fir.

The loading procedure for this test was the same as that used on the test of the OWJ roof systems. Deflections were measured after each increment of loading at midspan of the 12-foot span for three of the interior joists.

<u>3.2.2 Test Results and Discussion</u>. The wood joist roof is shown in Figure 3.24 supporting 24 and 36 inches of sand (200 and 300 psf). At each of these load levels, the roof was allowed to sit under load for several hours to observe changes in deflections due to creep. The deflection of the joists at midspan increased approximately 3 percent over

a 24-hour period with a loading of 24 inches of sand. When the load was increased to 36 inches of sand (300 psf), the roof supported the load for several hours before one of the joists failed. The split section of the failed joist can be seen in Figure 3.25. The failure occurred in the joist next to one of the edge joists of the roof.

The failure plane of the broken joist extended along the wood grain from near midspan on the underside of the joist to within 1-1/4 inches of the top of the joist at approximately 4 feet from midspan. Figure 3.26 shows a closeup of the broken joist after it was removed from the roof section.

Load-deflection curves for three of the joists for which deflection measurements were reported are shown in Figure 3.27. The surveyor's level used to measure deflections of the joists was moved after recording the deflections with 36 inches of sand on the roof. Therefore, no deflection measurements were recorded after the joist failure. The roof appeared to deflect considerably when the joist broke; however, none of the remaining joists showed any signs of failure after sitting under load for several more hours. The roof system was stiffer than predicted by the computed load-deflection curve, which could be due either to the modulus of elasticity of the fir joist being around 2,000,000 psi rather than the 1,760,000 psi given in handbooks or composite action being developed between the plywood decking and the joist. The stiffness of the roof system assuming composite action between the joist and the decking is twice that of the actual roof system. Therefore, if composite action did develop it was only to a small degree. These results are similar to those presented in Reference 5 where 14-foot-span wood joist floor sections were tested.

The results of this test show that a low-slope wood joist roof system designed using handbooks will support the 12 inches of soil cover required for radiation protection without additional supports. The floor systems tested in Reference 5 had an average maximum load capacity of 1.86 psi for the unreinforced specimen. This corresponds to a sand depth of approximately 32 inches. Wood joist floor systems of this type would also carry without additional support the 12 inches of soil

required for radiation protection. Before loading any wood floor or roof system, the joists and decking should be inspected closely for obvious defects in materials or construction.

3.3 REINFORCED CONCRETE ROOF SYSTEMS

There are numerous reinforced concrete roof systems. Upgrading of some of the more common types for which there are available test data will be discussed in the following paragraph.

3.3.1 Prestressed Double Tee Beams. These beams are frequently used for long span roof or floor systems where the underside of the beams serve as the ceiling of the area below. Manufacturer's catalogs show a variety of standard tee beams for different clear spans. These standard beams are modified by changing the quantity or strength of prestressing tendons to meet the loading requirements of a particular roof or floor system. Therefore, the load capacity of a particular roof constructed from prestressed tee beams must be computed using as-constructed properties from the building's blueprints.

Since prestressed double tee beams are designed in the same manner as other prestressed beam shapes, some of the results of tests on hollow-box or I-shaped prestressed beams would also be applicable to prestressed tee beams. Results of tests of hollow-box, I- and tee-shaped prestressed concrete beams are presented in References 6-9. Of particular interest are the results of hollow-box and I-shaped prestressed concrete beams presented in Reference 6. The beams were loaded with their design dead and live loads for 12 years before they were tested to failure. Some of the results of these tests are as follows:

1. The loading history of the beams had no apparent effect on the moment capacity of the beams.

2. The beams failed at loads that were 2.05 to 2.55 times their design dead plus live load capacity.

3. Maximum deflections prior to failure ranged from 1/52 to 1/64 of the clear span of the beam.

References 7-9 did not give the design loads for the beams that were tested. However, maximum deflections prior to failure were given

and were near those measured in Reference 6. Maximum deflection at failure was 1/66 of the clear span for the rectangular prestressed beams of Reference 7, an average of 1/75 of the clear span for the rectangular prestressed beams of Reference 8, and 1/25 and 1/76 of the clear span for the prestressed and posttensioned tee beams, respectively, of Reference 9.

From manufacturer's literature, the safe uniform load capacity of a double tee beam roof spanning 40 feet is 39 psf. The beam itself is 24 inches deep by 8 feet wide and weighs 57 psf. The design plus live load for this beam would be 96 psf (57 + 39 psf). Based on the results of Reference 6, the failure load of the beam would range between 197 and 245 psf. This is equivalent to adding 12 to 18 inches of soil weighing 100 pcf to the roof. The minimum failure load is the same as the desired soil loading for radiation protection; therefore, it would be unsafe to add 12 inches of soil to the roof without some type of additional supports.

The mass of the roof system itself is equivalent to 6-1/2 inches of soil. Therefore, the addition of 6 inches of soil, which the roof system could safely carry, would provide the desired roof mass for radiation protection. By measuring the midspan deflection during the placement of the soil, the soil depth could be increased until the deflection reached approximately 1 percent of the clear span. Midspan deflection at failure in References 6-9 varied from 1.3 to 4 percent of the clear span averaging 1.9 percent.

3.3.2 Two-Way Slabs. Results of tests on two-way reinforced concrete floor slabs apply also to two-way reinforced concrete roof slabs since the only difference between the two is the design loading. The failure load of the two-way floor systems tested in References 10-13 was more than 3-1/2 times the total design load for the floors. Actual collapse of the floor systems occurred at much higher loadings, in some cases as high as 10 times the total design load (Reference 12). Multibay models containing nine floor panels along with the associated beam and column support system were tested in References 10 and 13. In both cases, one with a reinforced concrete frame (Reference 10) and the other

with a steel frame (Reference 13), the frame survived the test without collapsing.

The ratio of failure load to total design load may be higher for roof slabs than for floor slabs due to the low design load requirements for roofs and the minimum requirements of the ACI Building Code (Reference 14) concerning slab thickness, reinforcing percentage, and reinforcing spacing. A typical two-way roof slab spanning 18 feet in each direction would be a minimum of 5 inches thick according to the ACI code requirements. The slabs dead load would be 62 psf and assuming a design live load of 20 psf the total design loading would be 82 psf. Adding 12 inches of soil gives a total roof load of 182 psf. Based on the results of Reference 10, the damage to be expected with this loading is the initial formation of yield-line cracks on the top surface and positive moment cracks on the underside of the slab. This type of damage occurred at a loading of approximately two times the total design load of the floor system tested in Reference 10. Therefore, two-way reinforced concrete slab roofs or floor systems will safely support the 12 inches of soil necessary for radiation protection. As with the double tee beam roof system, the mass of the roof itself can serve either as part of the required mass for radiation protection, thereby reducing the added soil depth, or as extra mass giving the shelter a higher protection factor.

3.3.3 Flat Plate Construction. Flat plate construction is commonly used where minimum floor-to-floor heights are required such as apartment complexes. The flat plate is essentially a flat slab without drop panels and column capitals at the column-slab connection. Large shear loads occur at the column-slab connection causing the floor system to fail in punching shear at the column. This is a brittle type of failure that occurs without warning.

Tests on flat plate structures are described in References 15 and 16. In these tests, failure occurred by punching shear of the column-slab connection at approximately 2.3 times the total design load for the structure. A typical flat plate roof system spanning 18 feet in each direction would be a minimum of 6 inches thick, according to the

requirements of the ACI code (Reference 14). Including the weight of a built-up roof, the design dead load would be approximately 85 psf. Roof live loads are in the range of 20 psf, giving a total design loading of 105 psf. Based on the results presented in References 15 and 16, the failure load for this slab would be around 240 psf. The addition of 12 inches of soil to the roof increases the roof loading to 205 psf or higher depending on the density of the soil in place on the roof. Since flat plates fail catastrophically, it would be unsafe to load the assumed roof system described above with 12 inches of soil. The addition of supports to this type of roof system could cause premature failure; therefore, it is not advised. However, the mass of the roof when included as part of the mass required for radiation protection would reduce the depth of soil required for radiation protection substantially. Six inches of concrete is equivalent to about 9 inches of soil. Therefore, the addition of 3 to 6 inches of soil would be safe and with the mass of the roof itself would provide radiation protection equivalent to 12 inches of soil on a lightweight roof system such as the wood joist or steel OWJ roof systems.

3.3.4 Flat Slabs. Flat slabs are often used where large live or dead loads are to be carried such as in parking garages, warehouses, or stores. Design methods for the flat slab evolved out of practice that produced floor slabs containing less reinforcing than the more conventional two-way slab, the design of which was derived from theoretical considerations (Reference 17). Prior to the 1971 ACI Building Code, design methods for the two-way slab required that it be designed for higher moments than a flat slab when both systems were designed for the same dead and live loads. Therefore, the inherent safety factor of the flat slab is lower than that of the two-way slab.

One-fourth scale three-bay by three-bay models of flat slab floors were tested by Hatcher (Reference 15) and Criswell (Reference 18). The Hatcher model failed at 1.9 times the total design load and the Criswell model, which was a blast resistant design, failed at 1.8 times the total design load. Both models were designed for large live loads. Due to the low live load requirements of roofs, many aspects of their design

are governed by minimum requirements of the ACI Building Code. Therefore, the failure to design load ratio of roof slabs should be higher than for floor slabs. However, there have been no tests to verify how much higher. Based on the results of the floor slab test (References 15 and 18), it is recommended that the mass of flat slab roofs be included in the mass for radiation protection, thereby reducing the depth of soil to be added to the roof. If the roof has been designed for high live loads, such as automobile parking, the full 12 inches of soil could be placed. In that case, it is assumed that the soil loading would be carried as part of the roof's design live load.

3.3.5 Ribbed Slabs. In ribbed slab construction, metal or reinforced plastic forms are used to create voids in what would otherwise be a solid slab. The resulting slab is designed for one- or two-way action. For one-way action, the slab is designed as a series of closely spaced tee beams. For two-way action, the slab is designed as a two-way or flat slab, where, in either case, the underside has a waffle-like appearance.

A series of tests to failure of a two-way ribbed slab roof system were conducted on the Rathskeller structure (Reference 19) located at the New York World's Fair. This was an unusual roof system in that it was designed for very large live and dead loads, 300 and 220 psf, respectively. Ratios of ultimate test load to the total design load were 2.0, 1.4, and 3.9, respectively, for the test with four interior panels loaded, three edge panels loaded, and a single interior panel loaded. The flexural strength of the roof was not reached in any of the tests. Ultimate strength was governed by shear at the interior and edge columns before a yielding mechanism could develop completely. Normally, a ribbed slab roof would be designed for a total load of 70 to 80 psf (50 to 60 psf dead load plus 20 psf live load). Based on the results of the Rathskeller test, the ultimate load for the roof could be as low as 100 to 130 psf (1.4 times the total design load). Since 12 inches of soil would produce a total roof loading of 170 psf or higher, failure could occur. Therefore, it is recommended that this type roof system be upgraded by the addition of soil mass only if the mass of the roof is

included in the mass required for upgrading. This should decrease the necessary soil depth to 6 inches or less.

3.3.6 Arch and Shell Roofs. Examples of these roofs are arches, cylindrical shells, folded plates, hyperbolic paraboloids, and domes. These roofs are normally used to cover large areas without interior supports or for their aesthetic qualities. Arch and shell roofs are designed to carry loads by internal axial forces with little shear or flexural stresses. Unsymmetrical loading that would probably occur during upgrading could cause sufficient flexural and shear stresses to fail the roof. The shape of these roofs would also make upgrading difficult. Therefore, reinforced concrete arch and shell roofs are not recommended for upgrading. Structures having these types of roofs could be upgraded by constructing a false roof inside the structure that could support the mass required for upgrading.

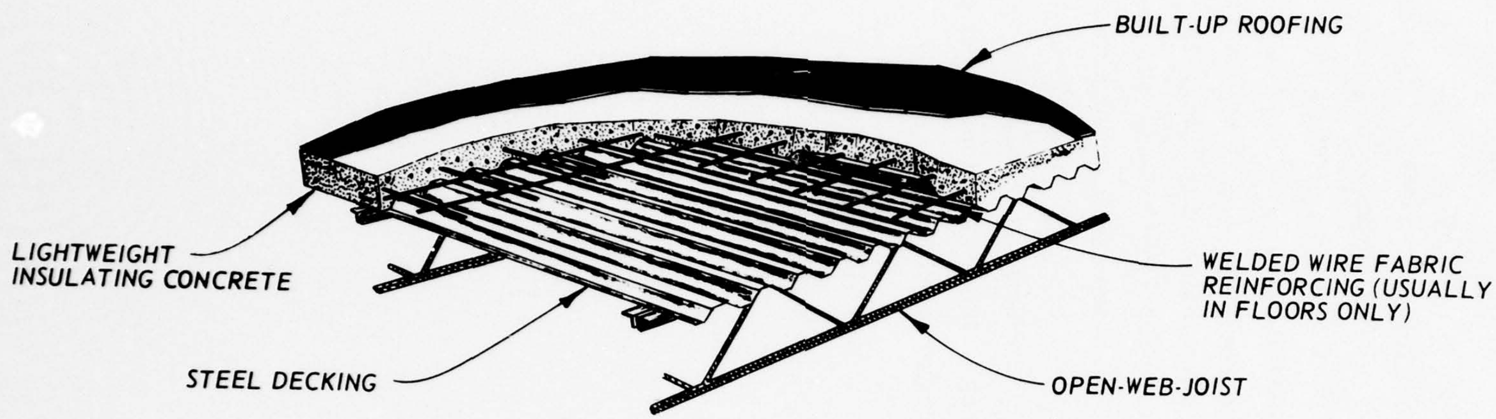


Figure 3.1 Typical OWJ roof system.

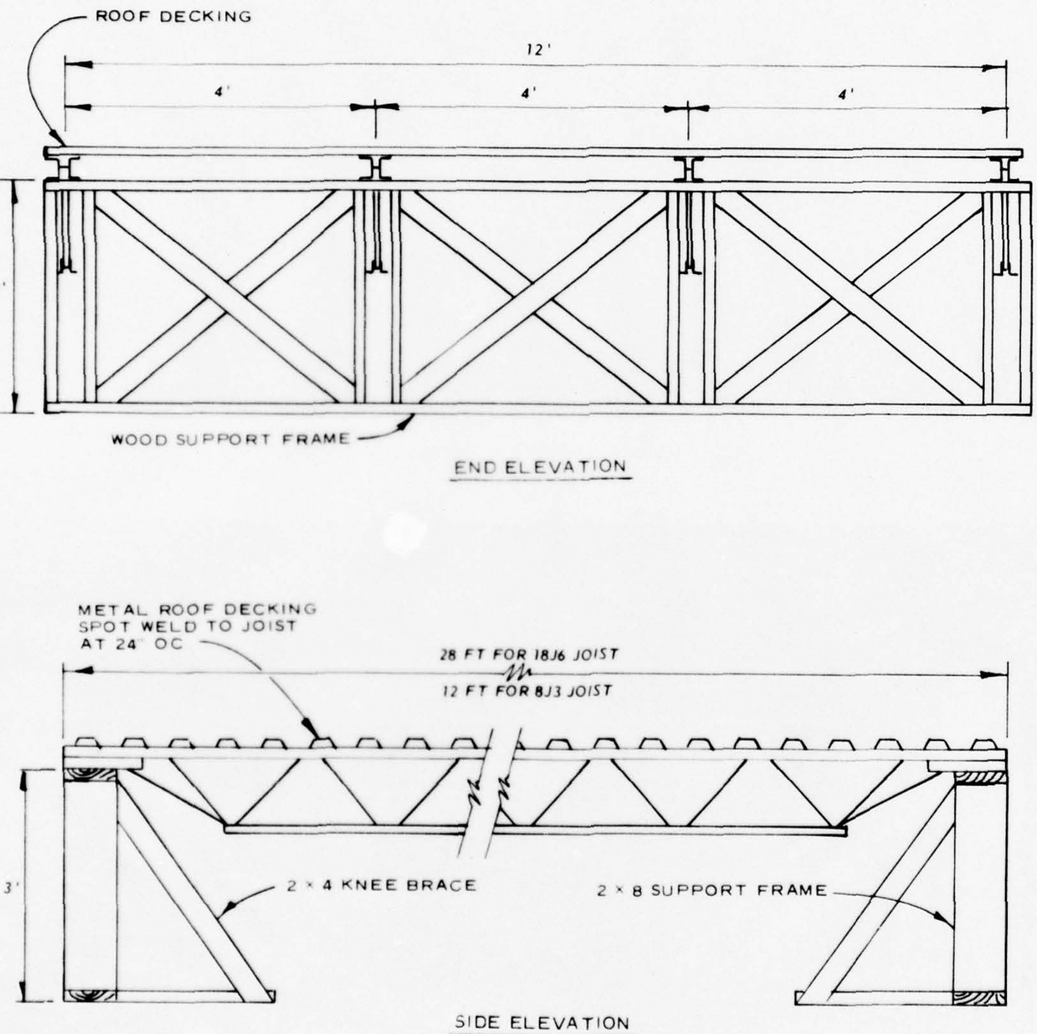
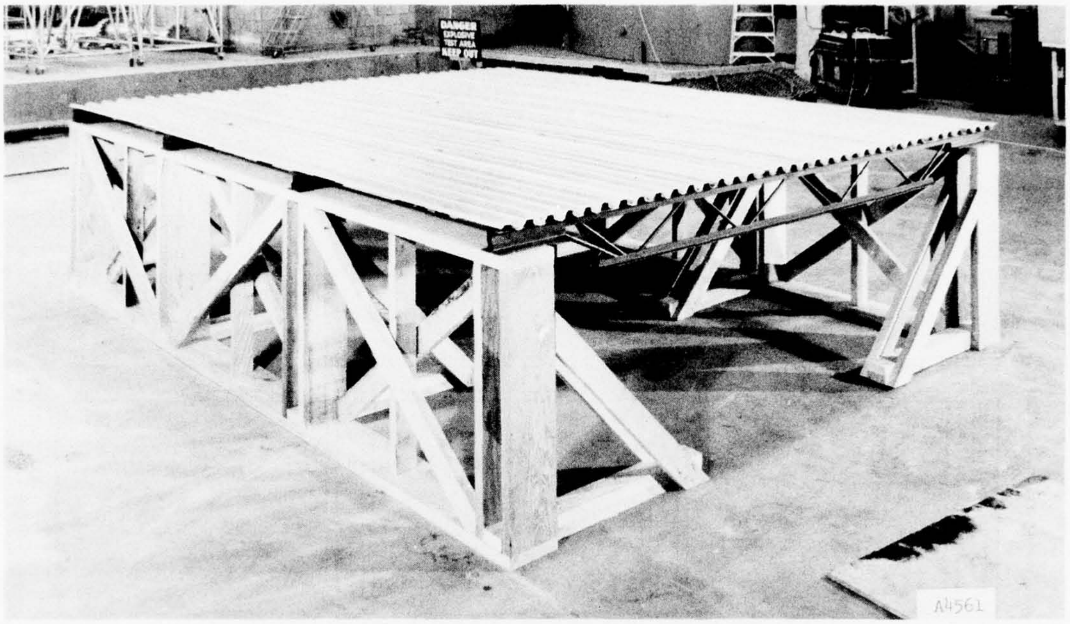


Figure 3.2 Test setup for OWJ roof systems.



a. View of metal decking on OWJ.

b. Underside view of OWJ roof system showing joists.

Figure 3.3 Ten-foot-span OWJ roof system.

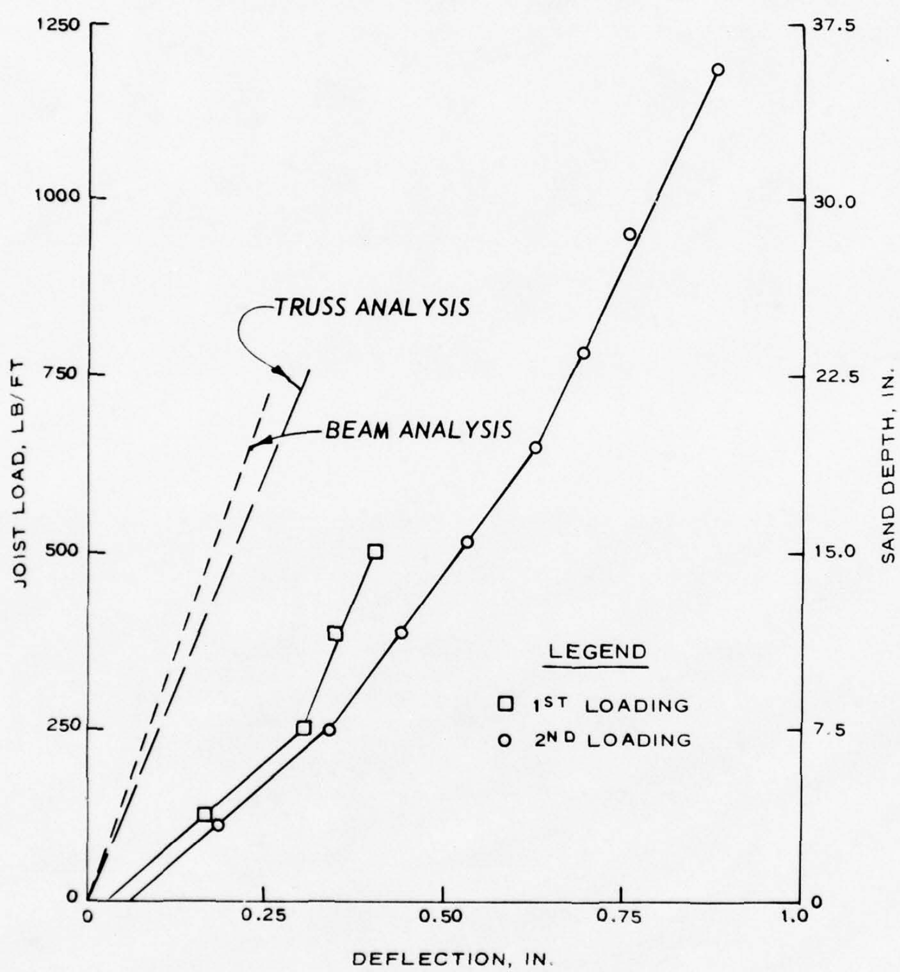
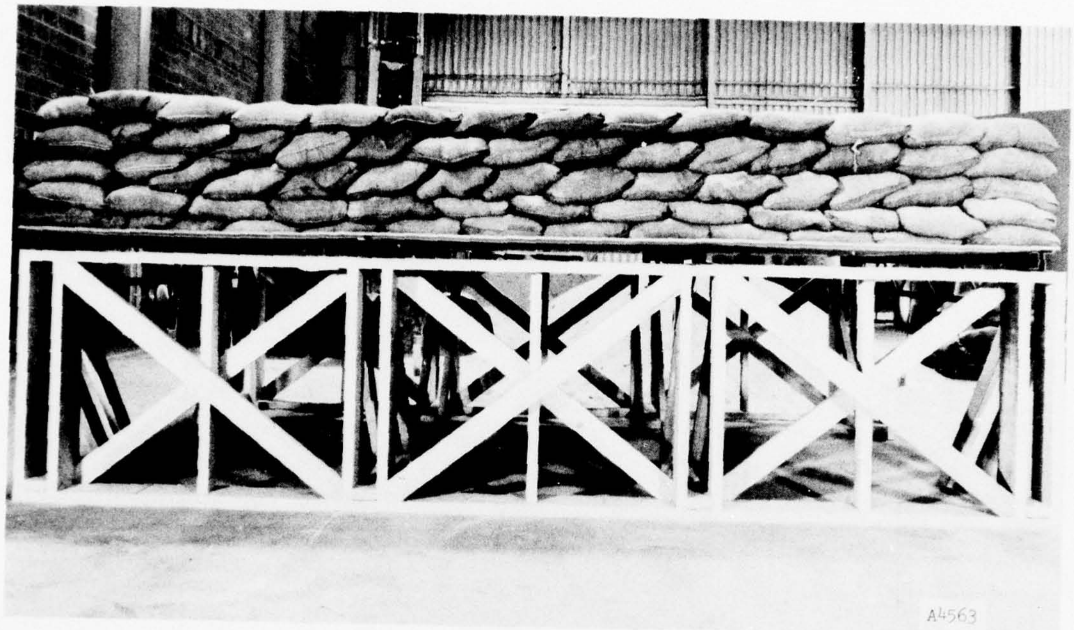
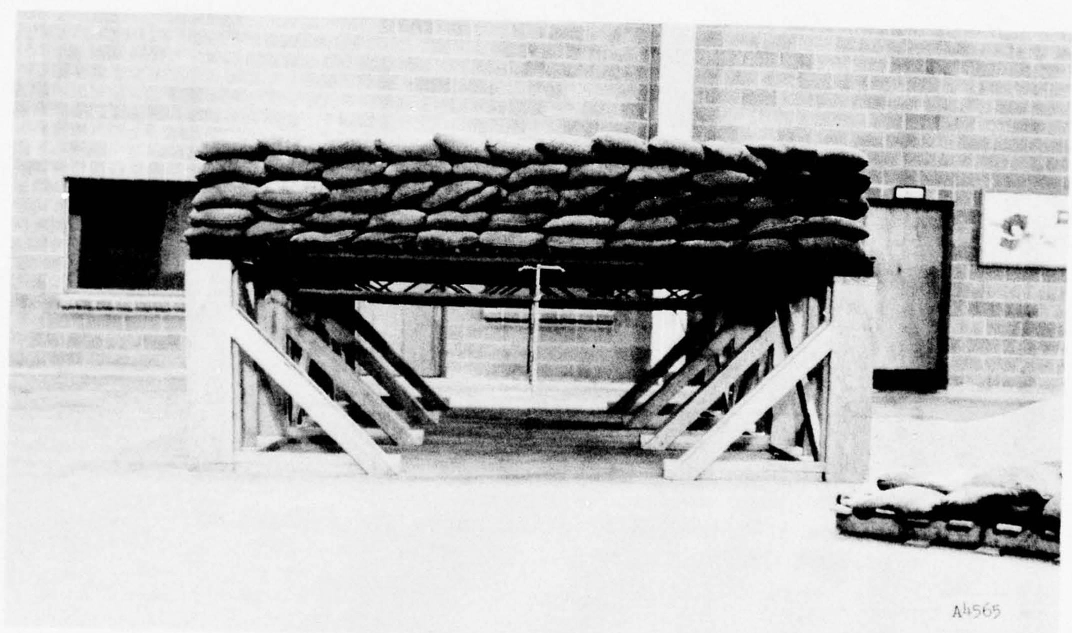


Figure 3.4 Load-deflection curve for midspan of 10-foot OWJ roof.

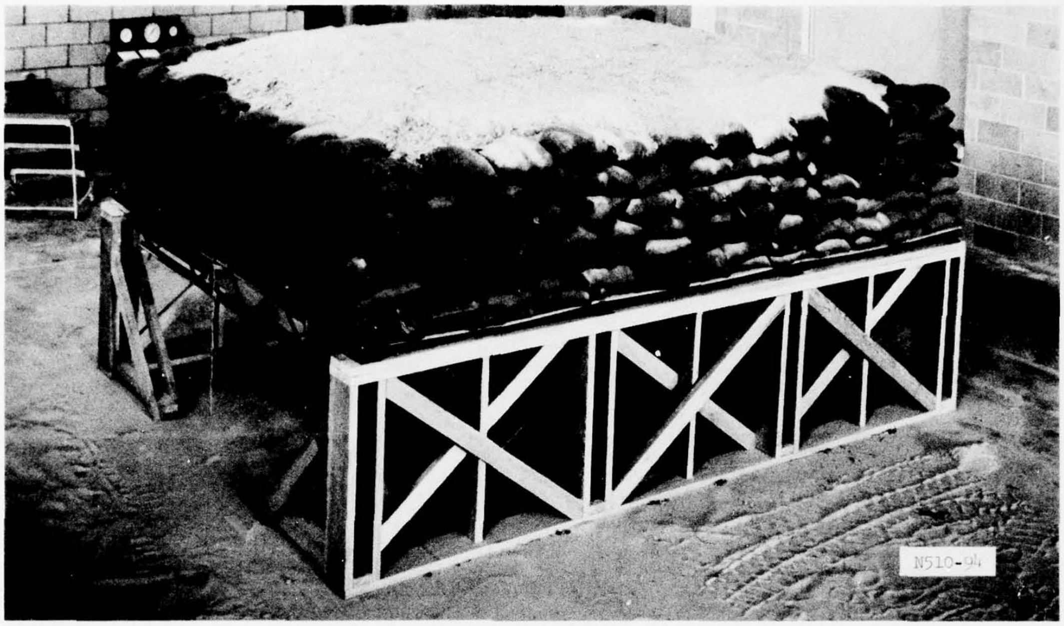


a. Side view.



b. End view.

Figure 3.5 Ten-foot OWJ roof system loaded with 20 inches of sand.



a. Overall view of loaded roof.

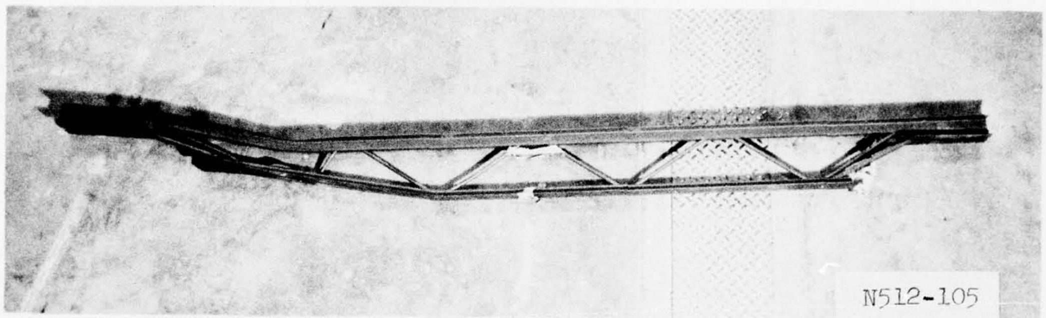
b. End view of roof deck and joists.

Figure 3.6 Ten-foot-span OWJ roof system loaded with 36 inches of sand.

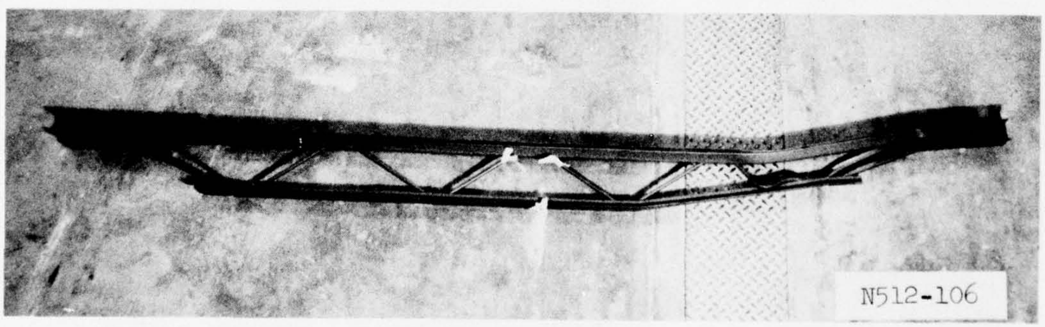
a. View of damaged joist from the west side.

b. View of damaged joist from the east side.

Figure 3.7 Initial buckling of the OWJ.

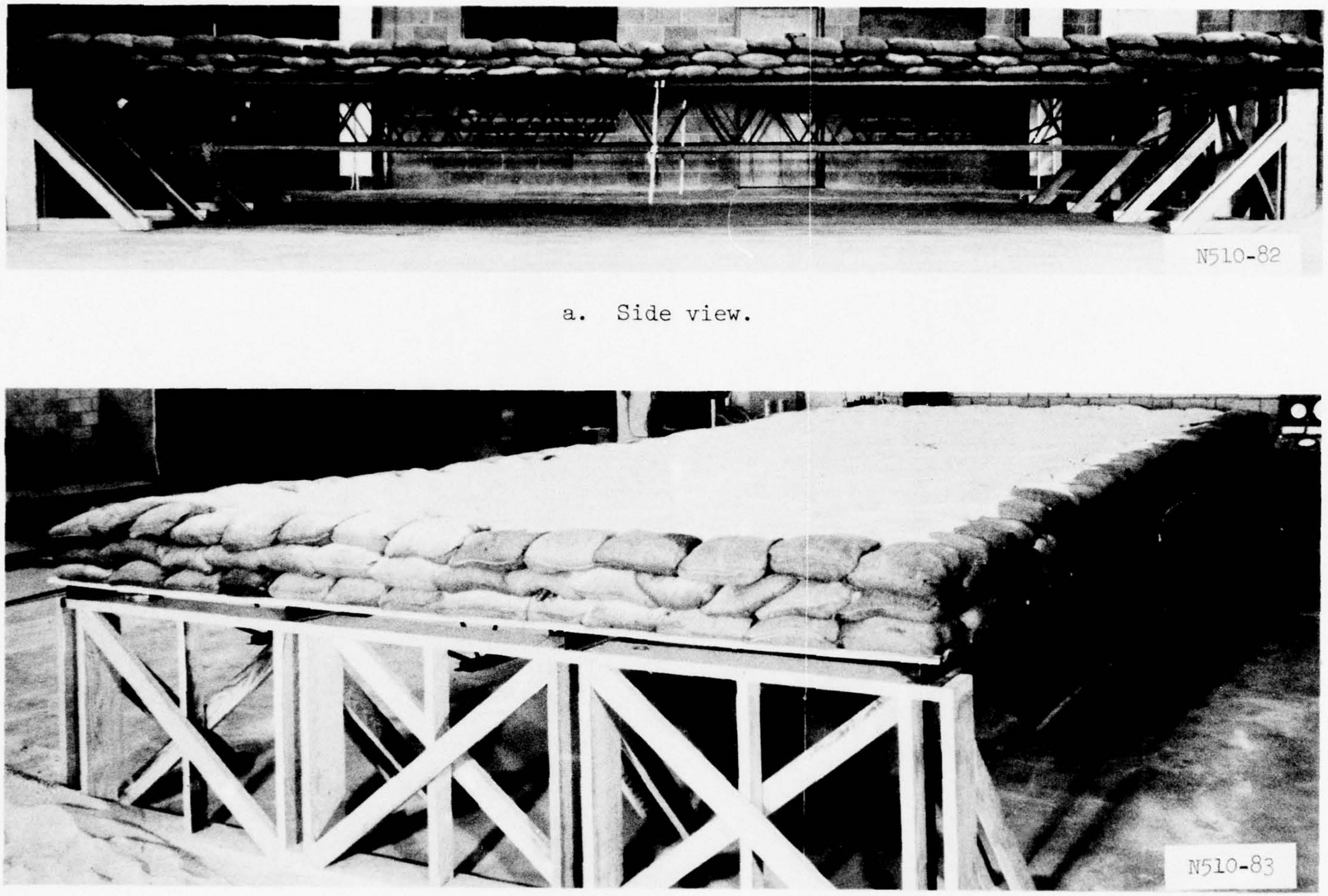


a. West side of joist No. 2.



b. East side of joist No. 3.

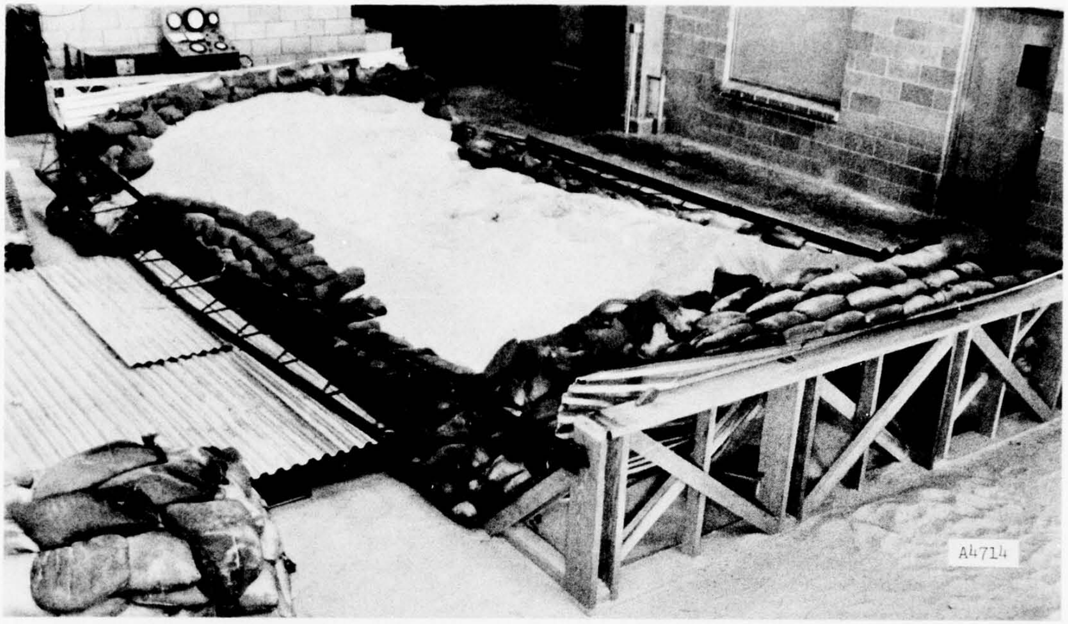
Figure 3.8 Overall view of damaged 10-foot OWJ.



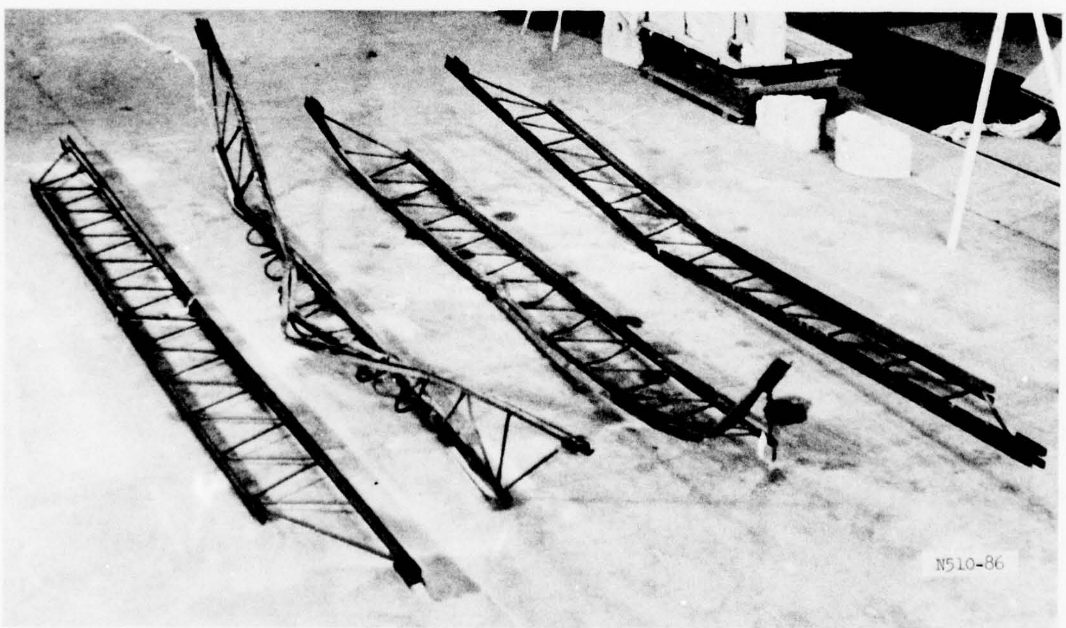
a. Side view.

b. End view.

Figure 3.9 Twenty-eight-foot OWJ roof loaded with 12 inches of sand.



a. Collapsed OWJ roof system.



b. Joist with sand and metal decking removed.

Figure 3.10 Twenty-eight-foot span OWJ roof after collapse.

a. Joist No. 1.

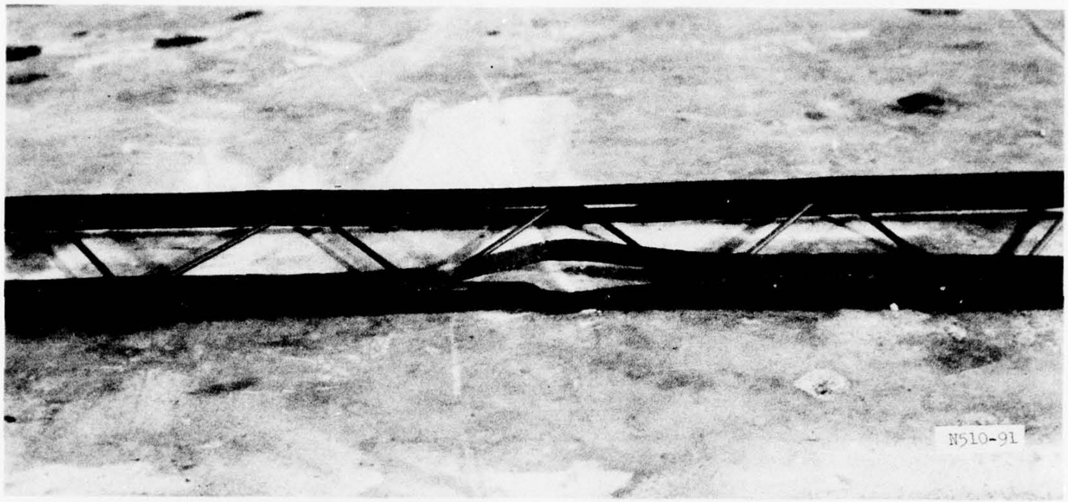
b. Joist No. 2.

Figure 3.11 Closeup of individual joist after failure (sheet 1 of 2).

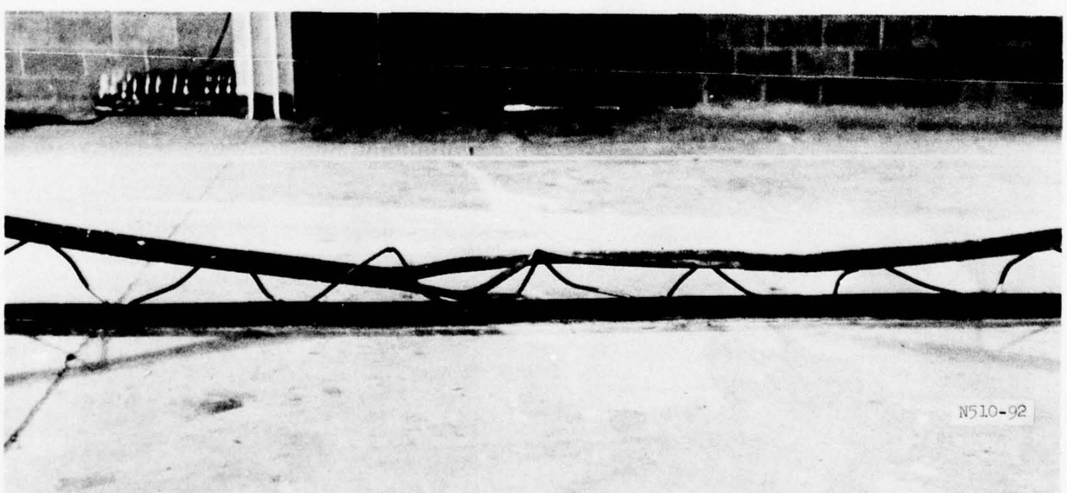
c. Joist No. 3.

d. Joist No. 4.

Figure 3.11 (sheet 2 of 2).



a. Buckled top chord of joist No. 2.



b. Central section of joist No. 3.

Figure 3.12 Closeup of major damage to joist Nos. 2 and 3.

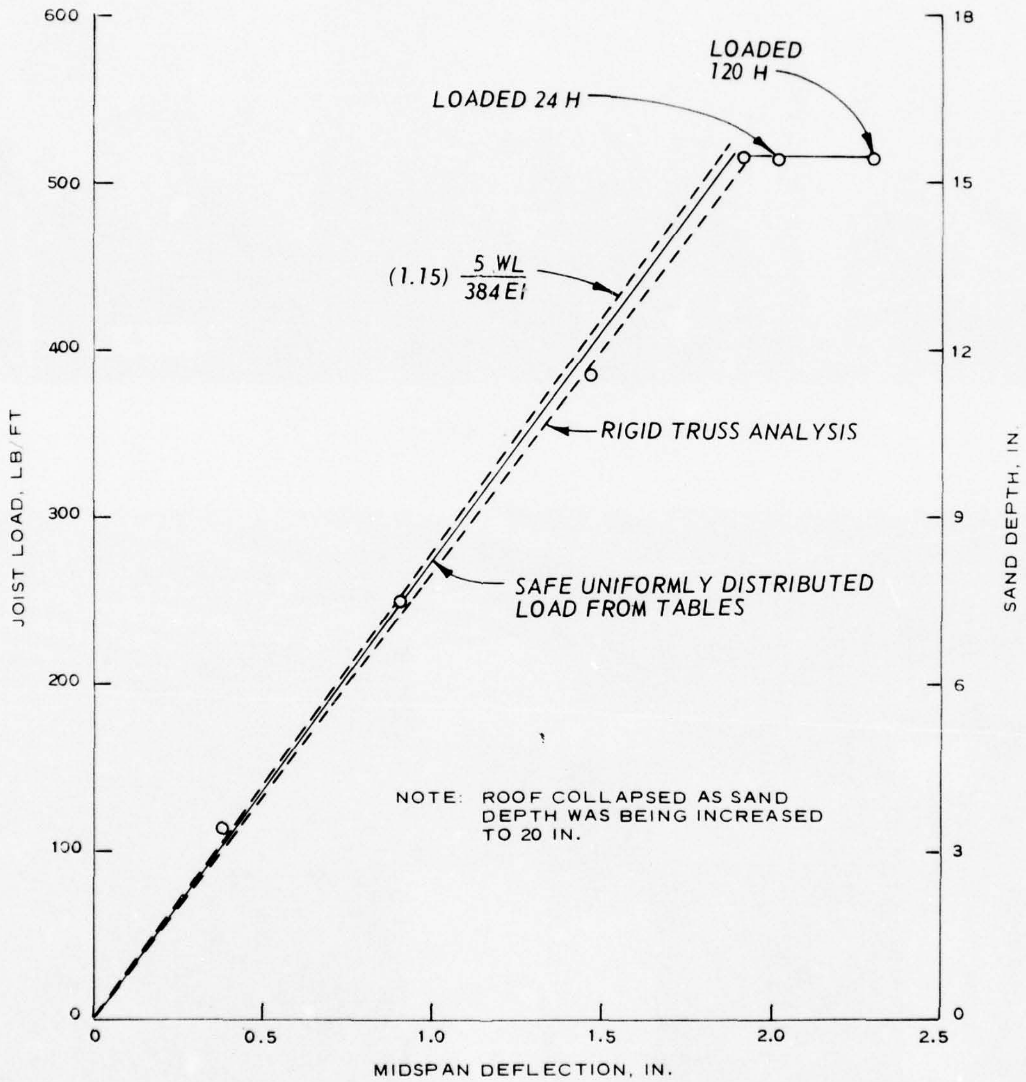
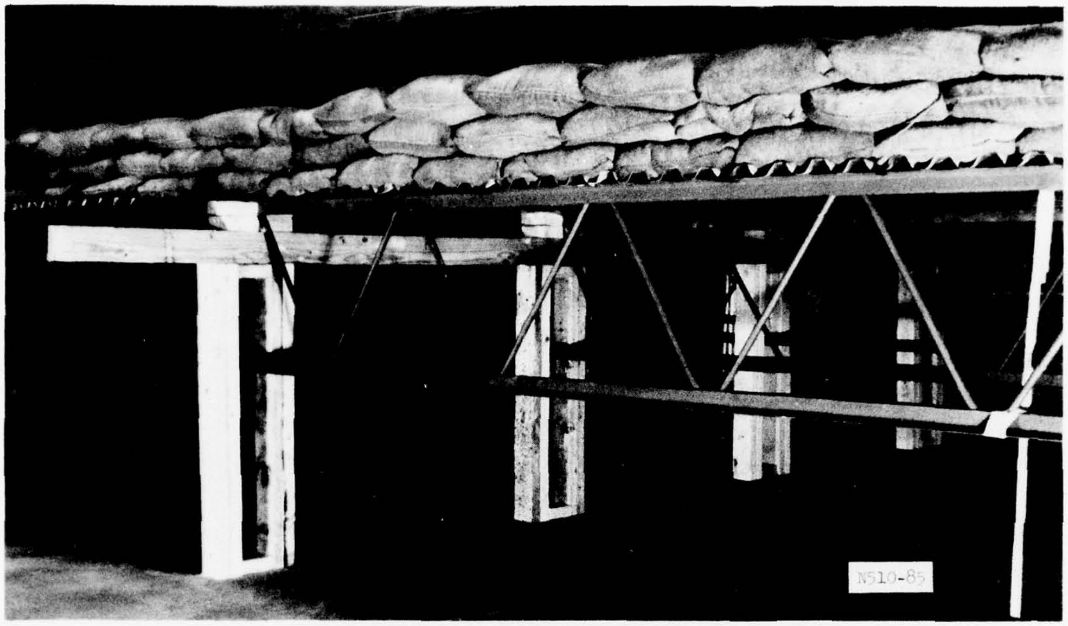
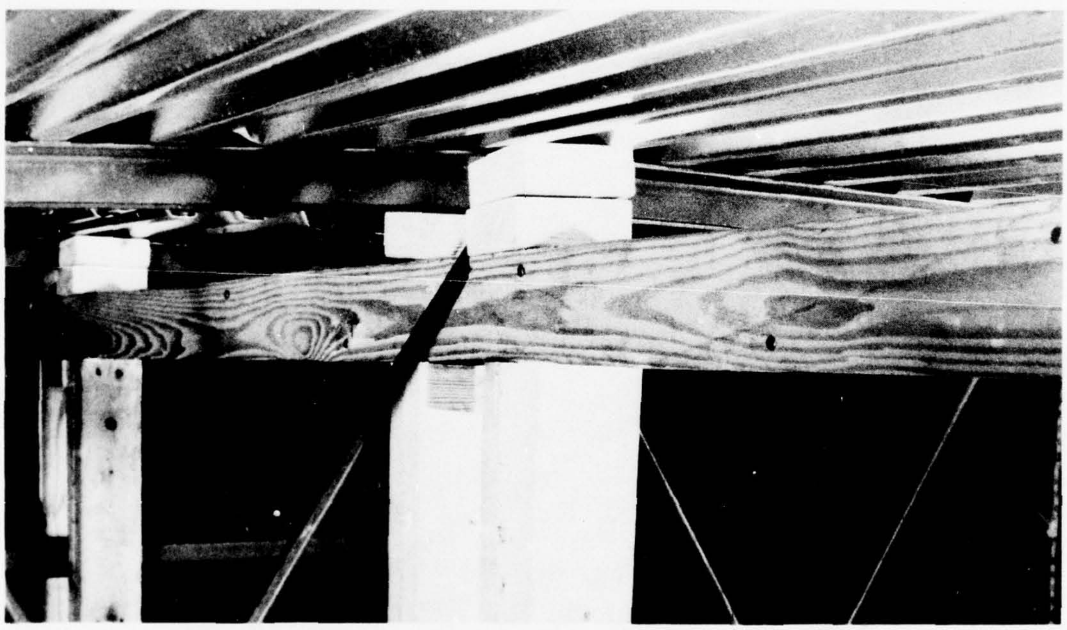


Figure 3.13 Load-deflection curve for 28-foot joist.



a. Support system.



b. Closeup of connection between OWJ and support.

Figure 3.14 Twenty-eight-foot OWJ roof with supports at midspan.

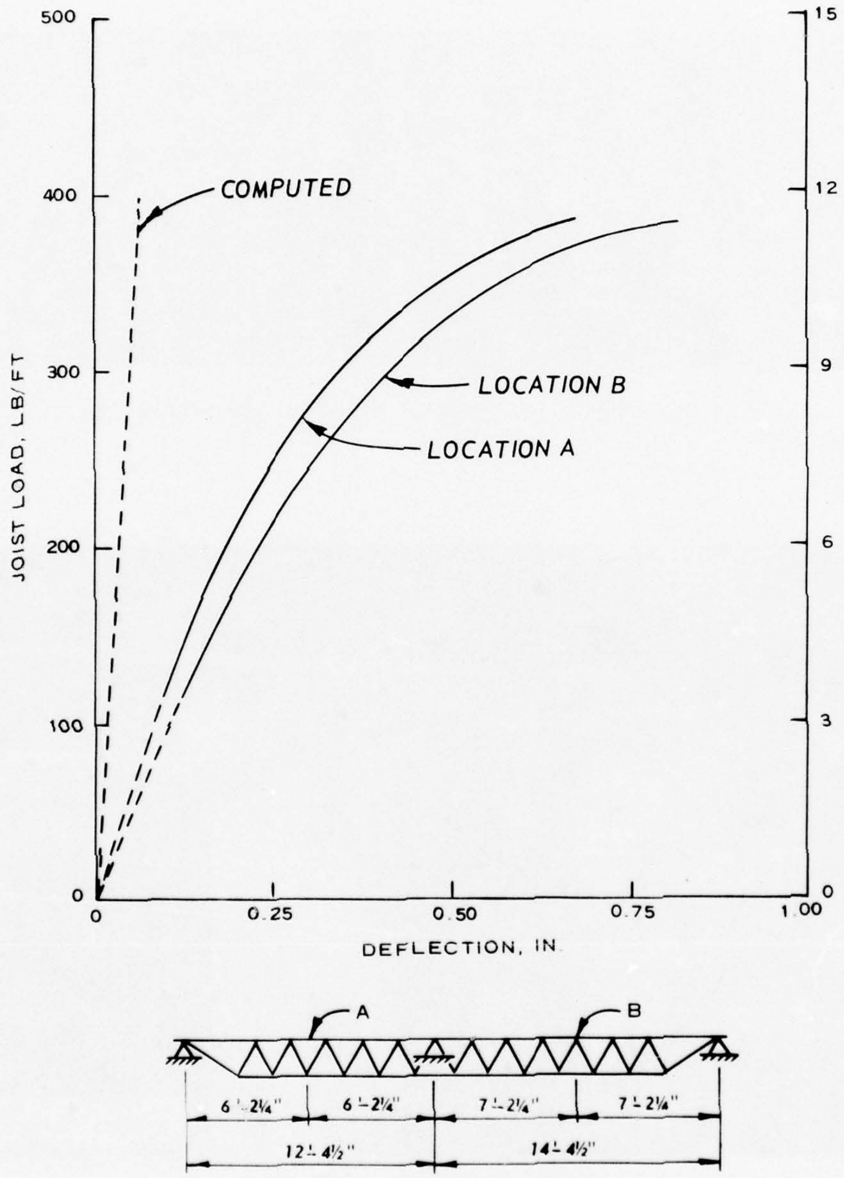
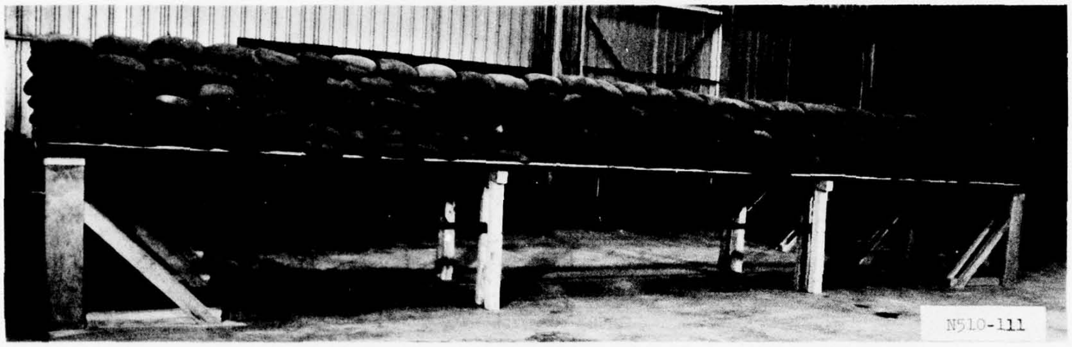


Figure 3.15 Load-deflection curve for 28-foot joist with midspan support.



a. OWJ roof with supports at one-third points.

b. Closeup of connection between OWJ and supports.

Figure 3.16 Twenty-eight-foot OWJ roof with supports at one-third points and a simulated 24-inch sand loading.

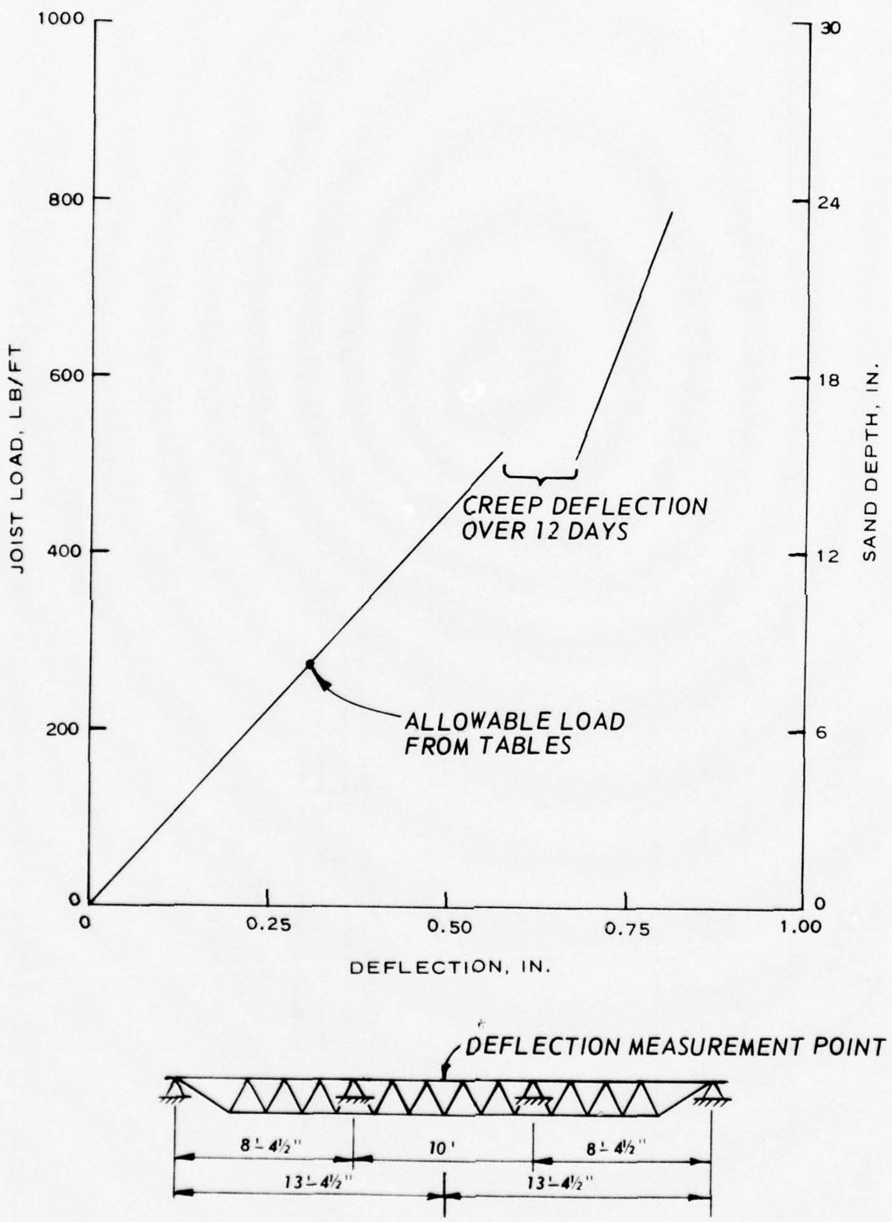


Figure 3.17 Midspan load-deflection curve, additional supports near one-third points, 28-foot joist.

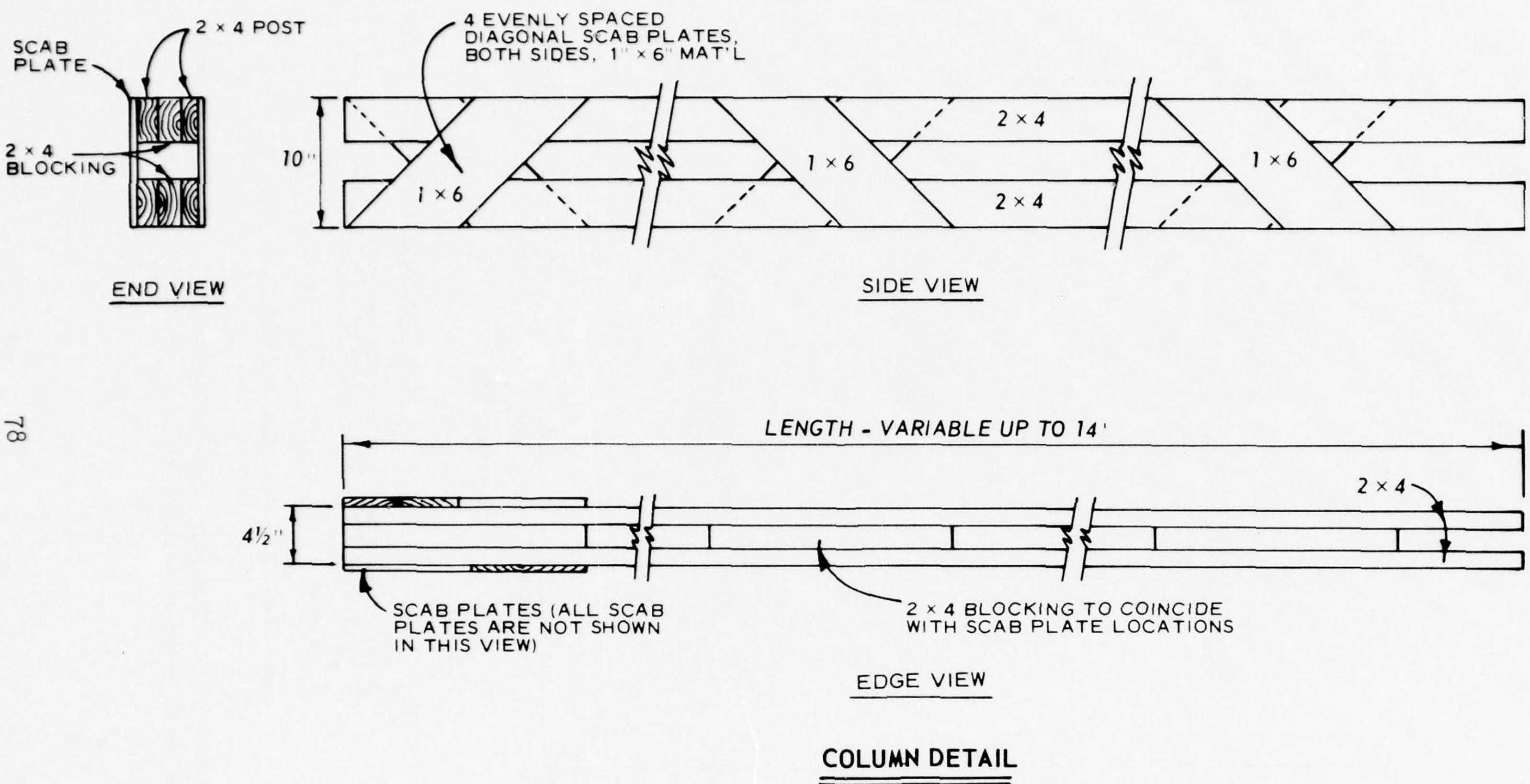
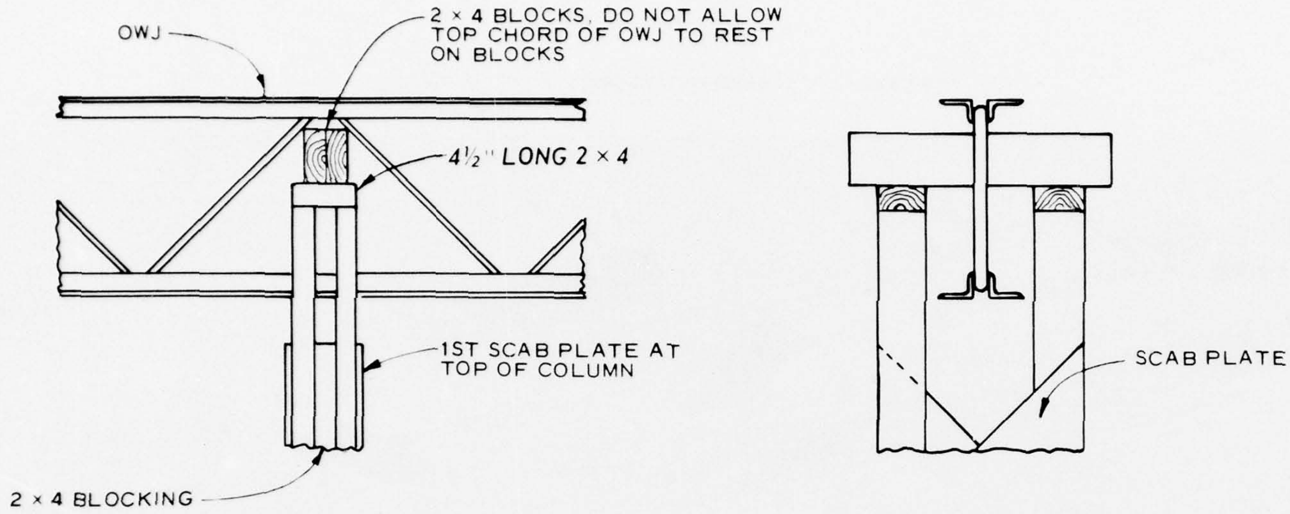


Figure 3.18 Support column and connection detail for OWJ roofs (sheet 1 of 2).



SIDE VIEW

END VIEW

CONNECTION DETAIL OWJ TO COLUMN

Figure 3.18 (sheet 2 of 2).

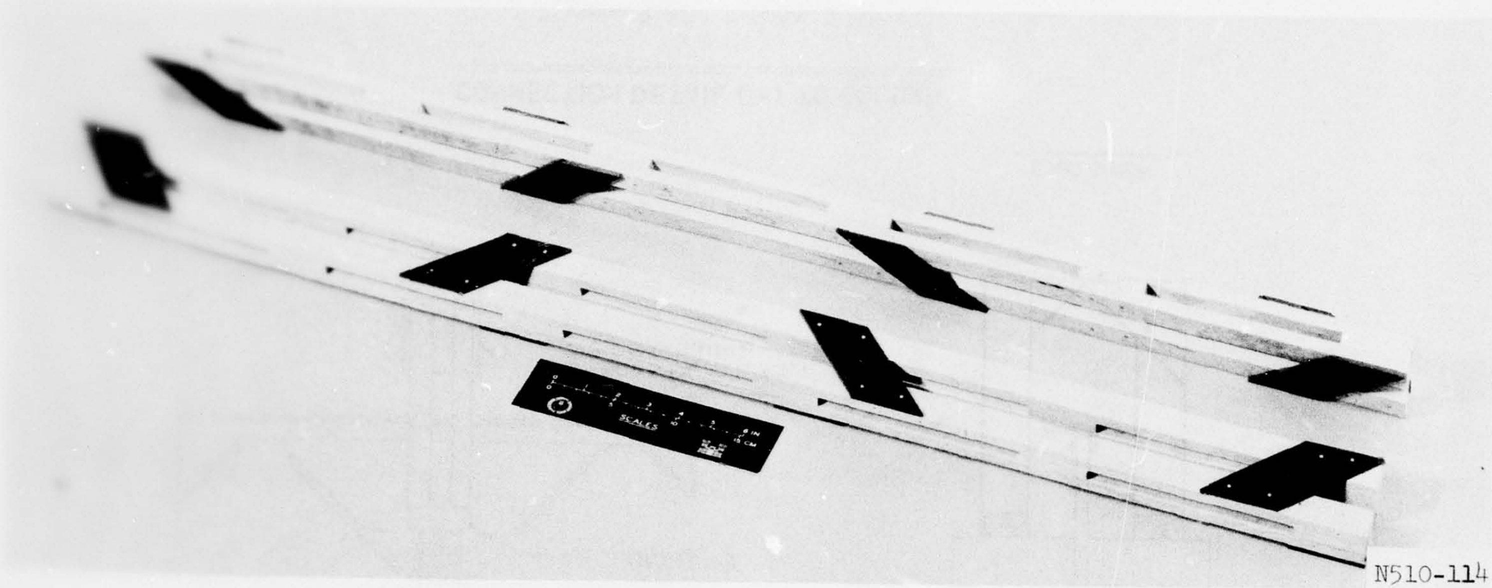


Figure 3.19 Model columns (1/4 scale).

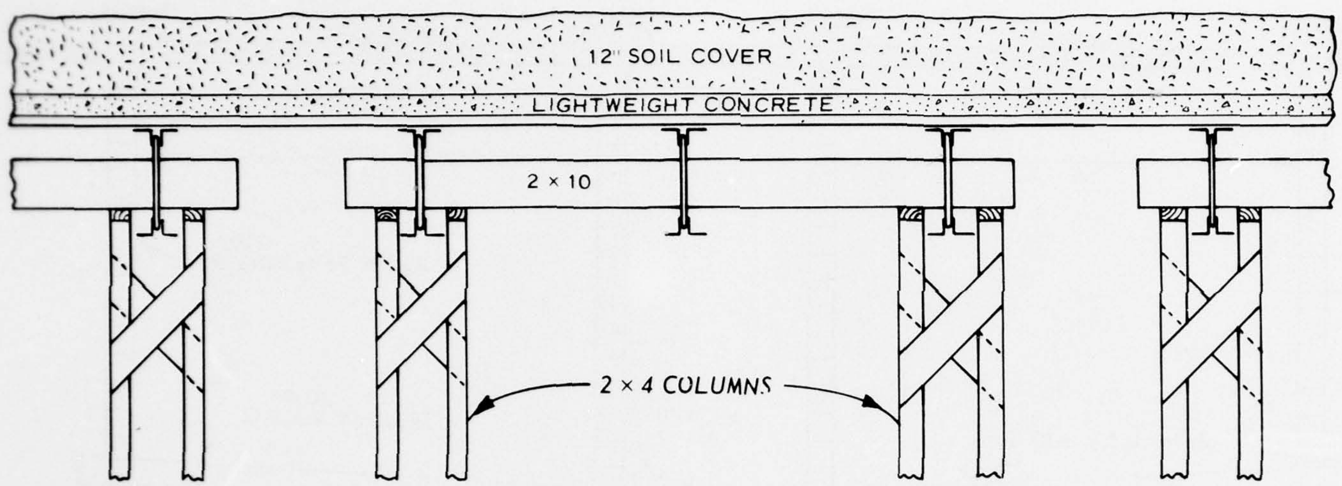


Figure 3.20 Layout for increased column spacing.

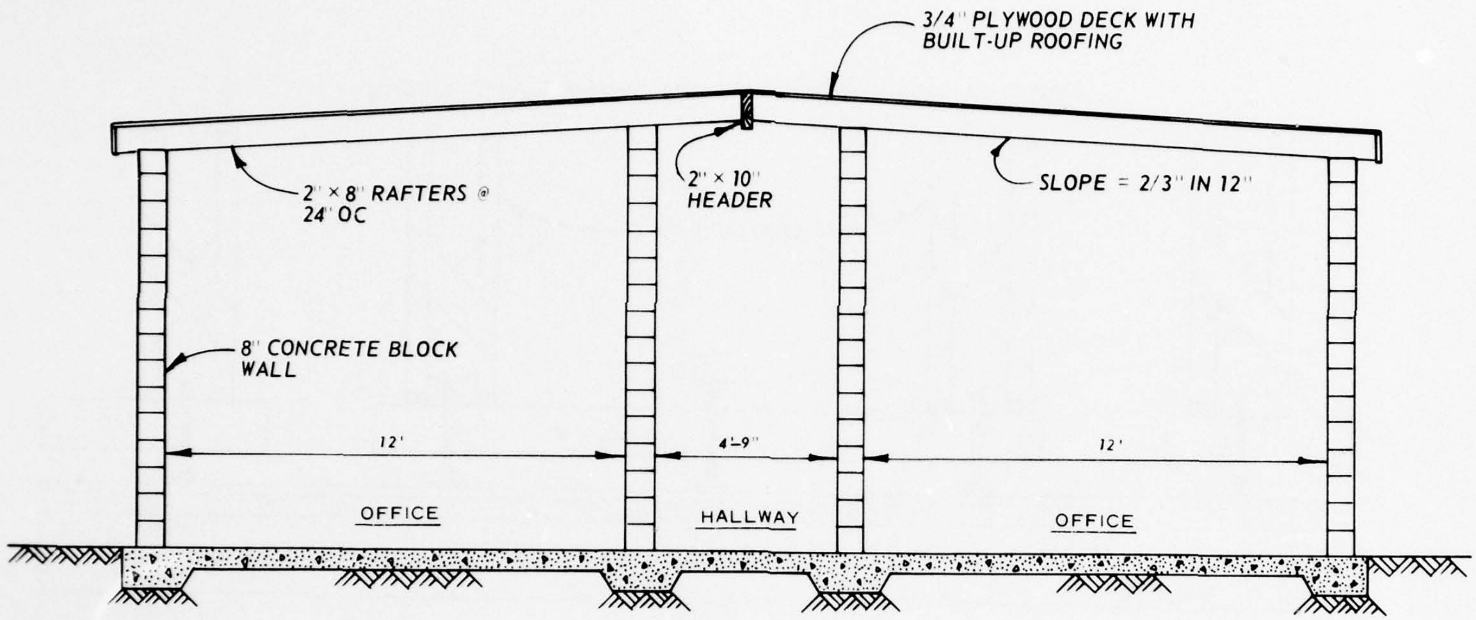


Figure 3.21 Typical low slope wooden roof system.

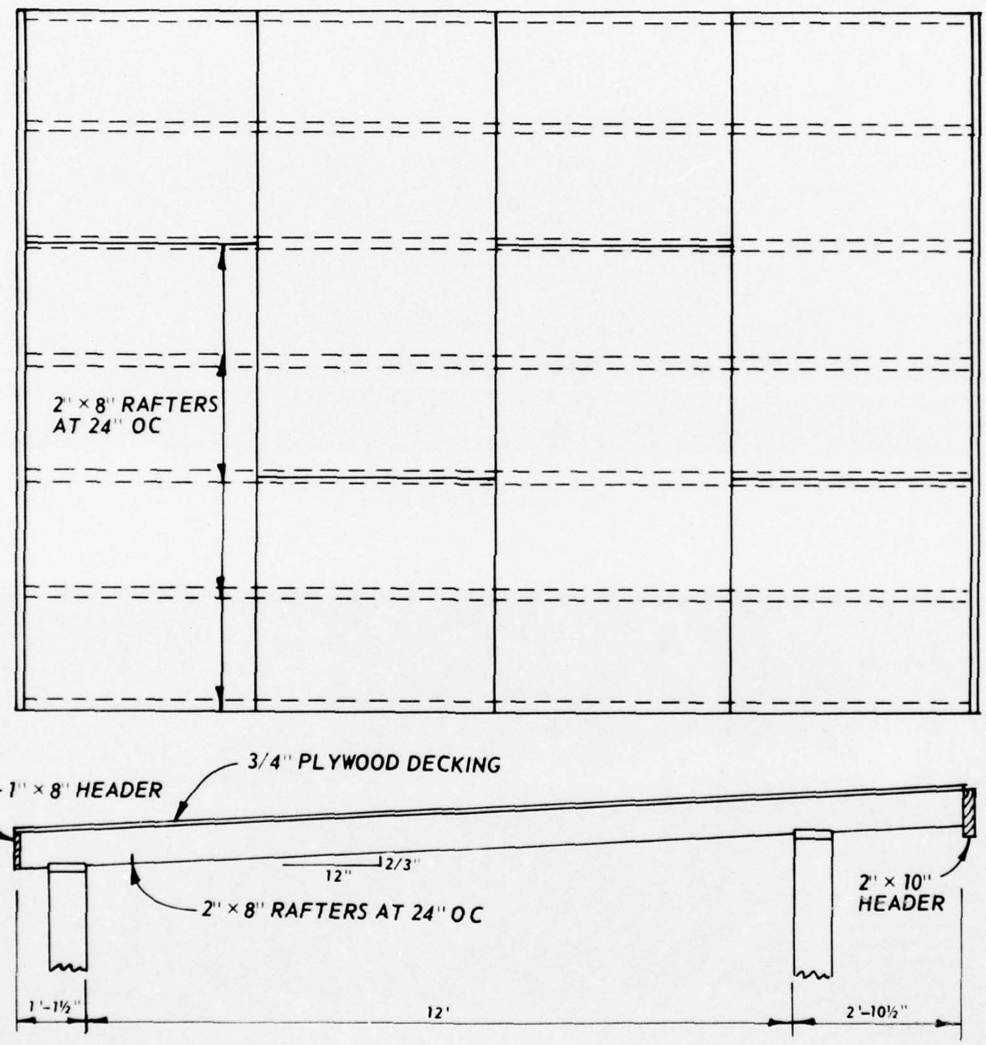
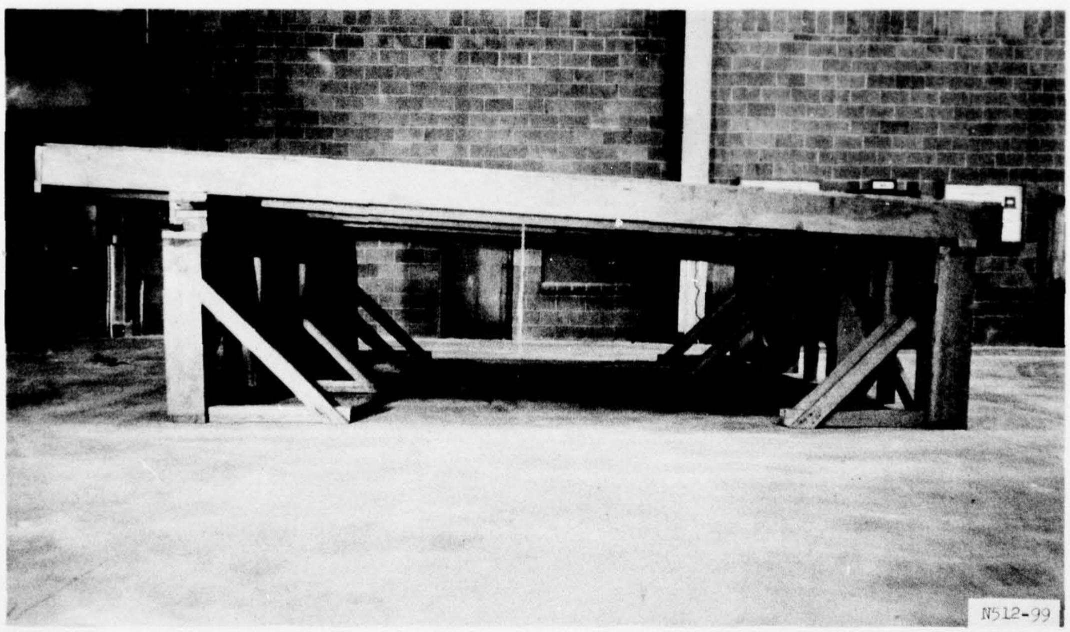


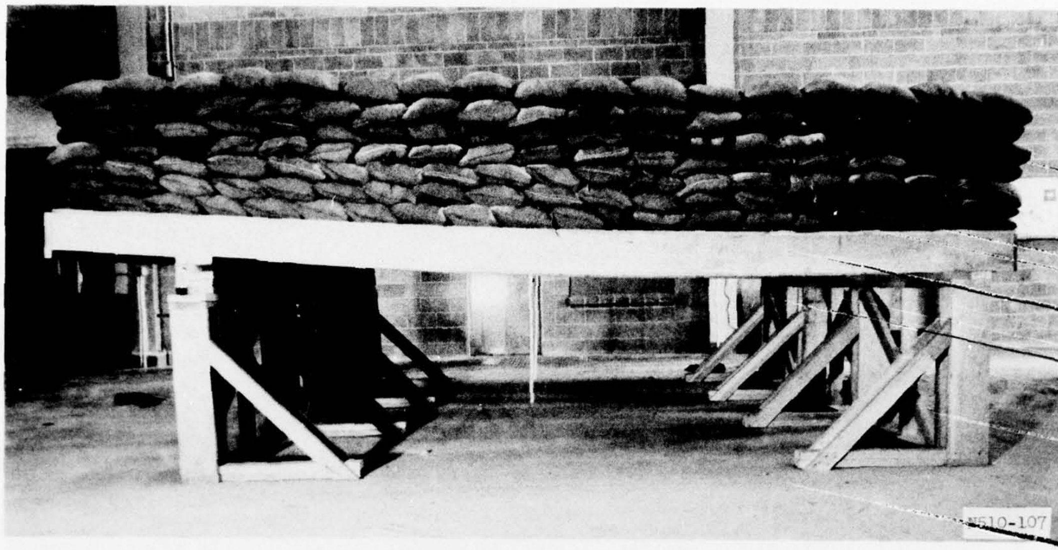
Figure 3.22 Plans for wood joist roof system.

a. Oblique view.

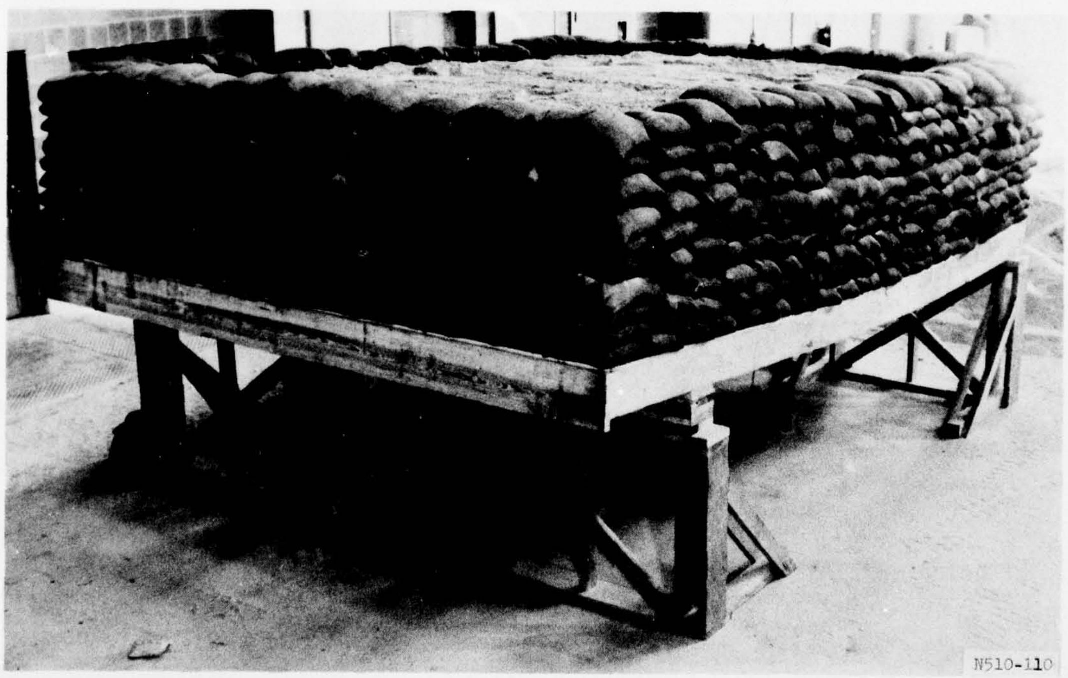


b. Side view.

Figure 3.23 Twelve-foot-span wood roof.



a. Edge view with 24 inches of sand on roof.



b. Oblique view with 36 inches of sand on roof.

Figure 3.24 Sand loading on wood roof system.

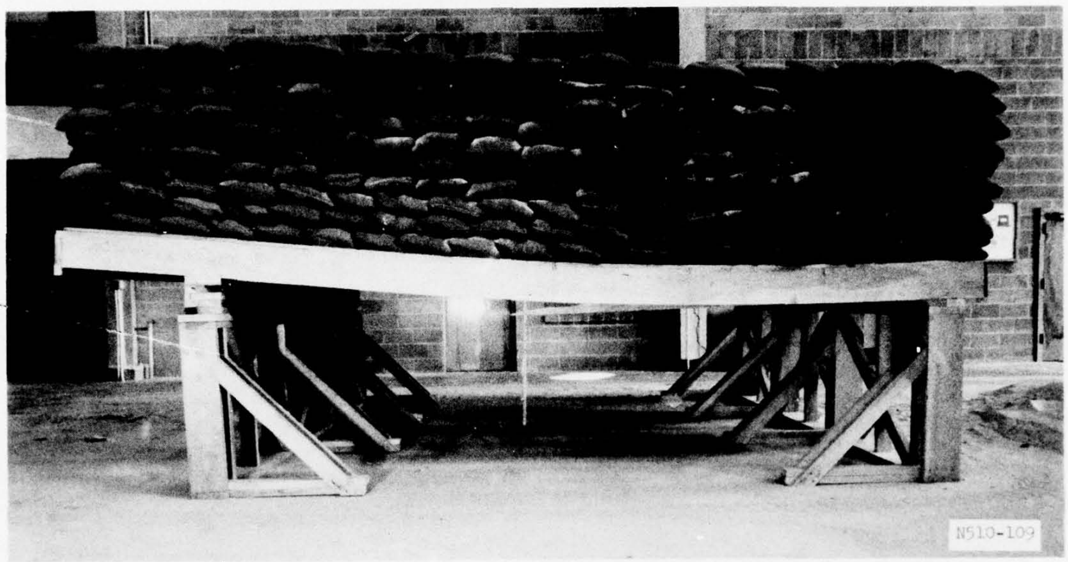


Figure 3.25 Wood joist roof after joist failure.

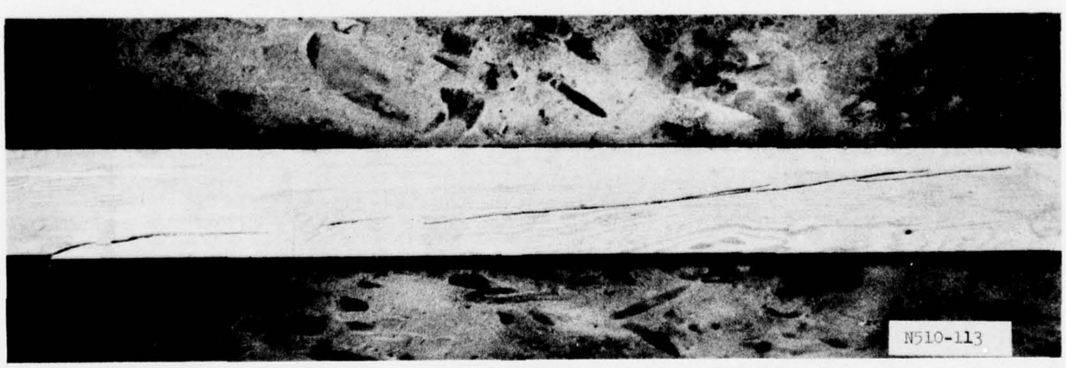


Figure 3.26 Closeup of broken joist.

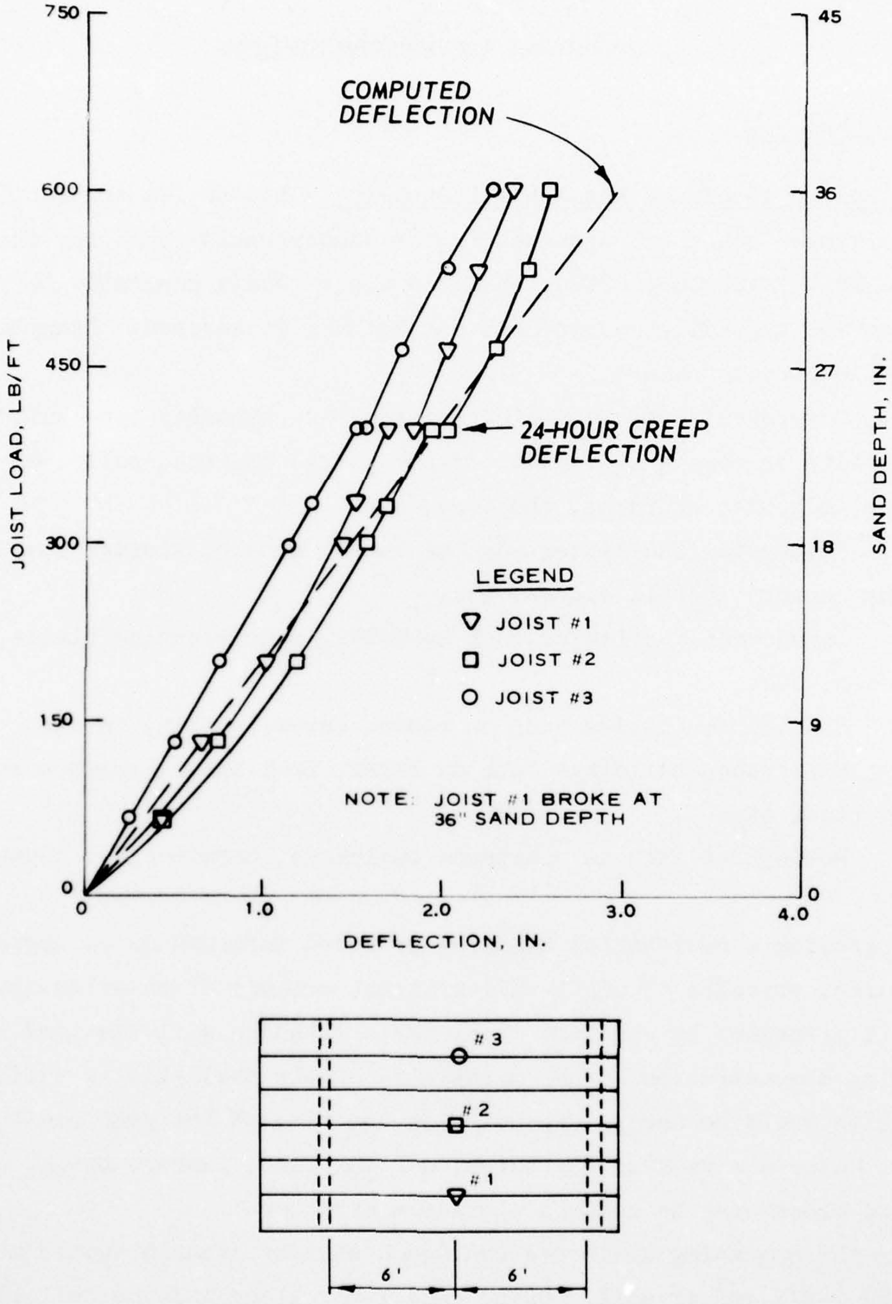


Figure 3.27 Midspan load-deflection curve for wood joist roof system.

CHAPTER 4

CONCLUSIONS AND RECOMMENDATIONS

4.1 CONCLUSIONS

4.1.1 Upgrading a Residential Dwelling. Due to its availability, a brick veneer house was upgraded in the demonstration test for this study. While structures other than residences would generally be selected for upgrading, residences can be used if desired. Examples of upgradable structures are:

1. Government owned, public, private, or community type buildings such as city or town halls, post offices, civic centers, police or fire stations, schools, churches, theaters, etc.
2. Commercial facilities such as banks, motels, storage companies, shopping centers, retail stores, etc.
3. Industrial facilities such as mills, manufacturing plants, warehouses, etc.
4. Special facilities such as mines, caves, parking garages, etc.
5. Farm/ranch buildings such as barns, tool sheds, bunkhouses, storage bins, etc.
6. Residences such as apartment buildings, dormitories, houses, etc.

Upgrading a residential dwelling provided information on upgrading procedures, strength of light-wall systems, methods of covering openings, and soil placement by hand and machinery. Although soil was used in the upgrading demonstration, other materials readily available in sufficient quantities could be used. Depending on the time of the year and the locale, materials such as hay bales, cotton bales, lumber, brick, or concrete blocks may be used to upgrade a structure.

In the upgrading demonstration, each shelter occupant would have to place 2-2/3 yd^3 of soil. Using hand labor alone and the soil placement times reported in the text, upgrading would require 461 man-hours (upgrading consisted of piling soil against the walls to a height of 6 feet above the floor slab and placing 12 inches of soil on the roof

of the structure). If only one half the shelter occupants are able to work, the minimum time required to upgrade the shelter is approximately 12 hours. Allowing for breaks and tiring of the workers, it is reasonable to conclude that the shelter could be upgraded by hand in the 2 or 3 days of expected crisis escalation. Mechanized movement of the soil would decrease the upgrading time.

4.1.1.1 Wall Upgrading. Walls of the house were upgraded by piling soil against them to a height of 6 feet above the interior floor level. The soil (loess) was allowed to assume its natural angle of repose which in this case was about 45 degrees. Hand labor using shovels and wheelbarrows, a front-end loader, and a crane equipped with a clamshell bucket were utilized in the soil placement. The most efficient soil placement method was the front-end loader. The walls were undamaged by the soil piled against them indicating that the soil forces were small.

From this test it can be concluded that all load bearing walls and curtain walls used in commercial construction can be safely upgraded using soil. The upgradable walls could be of the type: (1) all with masonary veneer, (2) concrete block, reinforced or unreinforced, or (3) timber stud walls.

Wall types not included in the above are glass curtain walls and corrugated sheet metal walls, which are used for warehouses or light manufacturing buildings. These two wall types can be upgraded with soil by covering the glass with a wooden wall or, in the case of corrugated metal buildings if the walls show signs of distress, soil can be piled on both sides of the wall simultaneously.

4.1.1.2 Covering Openings. Openings in the upgraded structure were covered with either 3/4-inch-thick plywood sheets or interior hollow-core doors from the house itself. Both materials were covered with plastic to keep out moisture from the soil. Over a period of several weeks moisture can cause delamination followed by failure of the hollow-core doors. With plywood, waterproofing is not as important. Large glass areas can be covered by spanning the area in the short direction with 2 by 4's and covering with plywood or wooden doors. Soil placed against the closure material will keep it in place; therefore, it is not

necessary to screw, bolt, or nail the closure material to the structure itself.

4.1.1.3 Roof Upgrading. The roof of the test structure was covered with soil using several methods that included hand labor alone and hand labor in conjunction with machinery. One section of the roof was load tested by increasing the soil depth to 24 inches. Under this loading, a web member of one of the trusses broke. The remaining trusses continued to support the load until the soil was removed several days later. The following conclusions were drawn from the roof upgrading test and analysis that followed:

1. A prefabricated wood truss roof used in residential construction can be upgraded safely by the addition of 12 inches of soil (100 to 120 psf) or equivalent mass to the roof.

2. A carpenter-built residential roof conforming to FHA specifications can be upgraded safely in the same manner as the prefabricated truss roof.

3. A soil roll fabricated from bed sheets as described in the test will hold 12 inches of soil on a roof having a slope of up to 3 in 12 (steeper roof slopes were not tested) even during rainy periods.

4. The roof can be upgraded by the shelter occupants without mechanical equipment. (Almost any type of mechanical equipment that will raise the soil to the edge of the roof will speed the upgrading process. This includes such equipment as front-end loaders, forklifts, conveyor belts, and material elevators. Cranes equipped with clamshell buckets or anything else that would produce impact loads on the roof should not be used.)

4.1.2 Shelter Under Slab. A shelter large enough for a family of four was dug under the on-grade reinforced concrete slab foundation of a residential dwelling. Construction of the shelter required 13-1/2 man-hours by workers accustomed to manual labor. The slab foundation spanned the shelter without damage. An unreinforced beam similar to the unreinforced perimeter beam used with some concrete slab foundations was load tested spanning a 4-foot-wide trench to determine the safety of constructing a shelter under a slab foundation of this type. The beam

supported several times the loading it would normally carry. Therefore, it is concluded that: (1) a shelter as described in the text can safely be constructed under a reinforced-concrete slab foundation that has reinforced or unreinforced perimeter foundation beams, and (2) that the shelter can be constructed by a man and his wife using tools and materials found in most homes during the expected period of escalating crisis (2 or 3 days). This shelter gives families living in houses without a basement an expedient shelter option for their homes. This type of shelter can also be constructed under a house having a conventional beam and joist foundation. Overhead radiation protection would then be provided by placing soil on the floor directly over the shelter area.

4.1.3 Roof System Test.

4.1.3.1 Steel OWJ Roofs. Load tests were conducted on 10- and 28-foot-span OWJ roofs patterned after the roofs over hallways and classroom areas, respectively, of a local school. The 10-foot-span OWJ roof supported 36 inches (300 psf) of sand prior to buckling of the web members near the joist ends. These joists are strong enough to safely support 12 inches of soil added for radiation protection. The addition of more than 12 inches of soil is not recommended due to the possible failure of the roof decking itself.

The 28-foot-span OWJ roof system failed at a loading of 16 inches (133 psf) of sand. Added supports at midspan and the one-third points were tested. Midspan supports caused the joist to be overloaded at the supports and are not recommended. The one-third point support system developed allowed the joist to safely support 12 inches of soil along with its design loading. Due to the design of OWJ's, it is imperative that the one-third point support system be attached to the joist as shown in Figure 3.18. An incorrectly attached support system will cause premature failure of the joist. Also, only OWJ roofs covered with metal roof decking should be considered for upgrading and even then upgrading should consist of the addition of a maximum of 12 inches of soil or an equivalent mass of any other upgrading material.

4.1.3.2 Wood Roof Systems. A flat roof designed to span 12 feet

and for a total load of 40 psf supported 300 psf prior to failure of one of the joists. The remaining joists continued to support the loading. Based on these results and previous tests on wood joist floor systems, it is concluded that engineer or architecturally designed flat wood roofs have sufficient overload capacity to be safely upgraded by adding 100- to 120-psf mass to the roof. Prior to upgrading the roof it should be inspected thoroughly and all defects repaired. Continuous inspection during upgrading is also recommended as an added safety precaution.

4.1.4 Reinforced Concrete Roof Systems Five reinforced concrete roof systems (prestressed tee beams, two-way slabs, flat plate, flat slab, and ribbed slabs) for which there were available test data were reviewed to determine the overload capacity for each system. Based on the results of the reviewed test data, the two-way slab roof system will safely support 12 inches of soil (100 to 120 psf) added for radiation protection. It would be unsafe to place 12 inches of soil on the other four roof systems unless additional structural supports were provided or the roof system was designed for large loads such as automobile parking or pedestrian malls. However, the mass of the roof itself could be included in the mass required for radiation protection. This would reduce the quantity of mass to be added for radiation protection by 40 to 50 percent, in which case the roofs could be safely upgraded.

Test data for arch and shell type roof systems were not reviewed. These types of roof systems normally carry their loading through axial forces with little flexural or shear stresses. Since an unsymmetrical loading is likely to occur during upgrading, these roof systems are not recommended for upgrading unless persons experienced in placing such structures underground are available.

4.2 RECOMMENDATIONS

Almost any structure can be upgraded. Some require only that mass be added to the walls or roof while others will require structural modifications. Therefore, it is recommended that for the host areas a list of upgradable structures complete with upgrading options and priorities

be prepared. This can be accomplished in the same manner that the present system of fallout shelters was developed. This would avoid shelter selection during a period of crisis and would provide an opportunity for engineered upgrading options to be developed for particular shelters if needed.

Based on the experience gained during the upgrading demonstration, each structure should be thoroughly inspected immediately prior to upgrading for defects in materials and construction. Any defects should be repaired and inspection continued during the upgrading process to detect any weak points in the structure.

In this study only a few of the more common types of the endless variety of roof systems were tested. Roof systems such as the OWJ roof can have their upgrading potential changed drastically by the type of decking used. For OWJ roofs, it is recommended that only roofs that use metal decking covered with several inches of concrete be considered for upgrading until further information is developed.

The conclusions reached for reinforced-concrete roof systems were based on previous test results for reinforced-concrete floors. The same design procedures are used for both floor and roofs. However, many of the minimum ACI Building Code requirements will apply to roofs due to their light design loads. Estimates for the overload capacity of roofs based on floor tests may be conservative. Actual roof overload capacities can only be obtained through tests.

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References: p. 94-95.

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This study was conducted in support of the Defense Civil Preparedness Agency's (DCPA) Crisis Relocation Planning (CRP) program in which existing structures will be upgraded to provide fallout shelters for a relocated population. A demonstration test was conducted in which a residential dwelling was upgraded by placing soil against the walls and on the roof of the structure. The shelter was large enough to house 80 people. Upgrading was accomplished partially by hand labor and machinery. The test showed that a conventional structure could be upgraded and that the shelter occupants using tools and materials found in most homes could if necessary upgrade their shelter during the expected 2- or 3-day period of crisis relocation preceding a nuclear attack.

Several roof systems were tested and others were analyzed based on previous test results for overloads that occur from upgrading. A system of added supports was developed that would allow steel open-web joist roofs to be upgraded. Flat wood roof systems were found to have sufficient overload capacity to be upgraded without added support. Upgrading in all cases consisted of adding 100 to 120 psf of mass to the roof. Some concrete roof systems such as the flat plate were found unacceptable unless the quantity of upgrading material was reduced. The two-way slab roof could be safely upgraded without modifications.

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